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SANTA CRUZ HARBOR SHOALING STUDY, SANTA CRUZ HARBOR, CALIFORNIA--ETC(U)
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SANTA CRUZ HARBOR SHOALING STUDY

FOR U.S. ARMY CORPS OF ENGINEERS
SAN FRANCISCO DISTRICT

MOFFATT & NICHOL, ENGINEERS

JUNE 1978

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SANTA CRUZ HARBOR SHOALING STUDY

SANTA CRUZ HARBOR, CALIFORNIA

(15)

Contract No. DACW07-77-C-0023

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Prepared for

U.S. ARMY CORPS OF ENGINEERS
San Francisco District

Prepared by

MOFFATT and NICHOL, Inc ~~ENGINEERS~~
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(11) June 1978

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PREFACE

This report was prepared by Moffatt & Nichol, Engineers under the direction of Dr. James R. Walker with assistance from Mr. Peter J. Williams and Mr. James W. Dunham. The interim dredging program was developed with assistance from Mr. William Herron and dredging consultant, Mr. Ogden Beeman. Laison with the San Francisco District, Corps of Engineers, was with Mr. Doug Pirie under the direction of Mr. H. Pape, Chief of Engineering. Col. John M. Adsit was the district engineer. The Santa Cruz Harbor Master, Mr. Brian Foss, dredgerman, Mr. Phil Stanfield and dredging inspector, Mr. Dave Dickson, provided assistance in developing and evaluating the interim dredging program.

Abstract

SYLLABUS

This study was made to define the littoral processes and resultant shoaling mechanism of Santa Cruz Harbor entrance channel and to develop and evaluate alternative methods of mitigating the shoaling effects. During the winter months, the Santa Cruz Harbor entrance channel has shoaled almost completely since its construction in 1962. The channel was maintained by annual dredging in the late winter or early spring until 1977 when the dredging procedure was changed. The study involved analysis of shoaling mechanisms and dredging procedures. A new two-year iterim dredging procedure was developed for the 1977 through 1979 winter seasons. The concept was to dredge the channel periodically in phases each winter and thereby keep the harbor open to navigation most of the time.

The littoral processes study led to the conclusion that the net littoral transport rate was 300,000 to 500,000 cubic yards per year from west to east. The wave energy-flux method of estimating littoral transport rates indicated that a potential for a reversal in transport exists but that the shoreline east of the harbor was so oriented that reversals in transport during the winter were not likely to cause significant shoaling in the channel. Observation of shoaling patterns, and analysis of dredging records and condition surveys indicated that about 100,000 cubic yards per year drifted around the head of the west jetty and formed a tip shoal in the entrance channel. Other mechanisms, accounting for an additional 50,000 cubic yards per year, include leakage of sand through the voids in the west jetty, updrift movement or reversal in littoral transport, wind transport over the west and east jetties, onshore transport of bottom material drifting outside the harbor entrance, leakage through the east jetty, and tidal current transport. Wave energy that propagates through the harbor entrance distributes littoral drift along the sides of the channel and farther into the channel.

A considerable quantity of littoral drift is believed to bypass the harbor offshore by natural processes. This quantity is estimated to range from 175,000 cubic yards per year to 375,000 cubic yards per year, varying with the wave climate and dredging procedures.

cont'

CONT

Sixteen alternative solutions that could mitigate channel shoaling were developed and analyzed. They were classified into bypassing, channel-maintenance and structural categories. The solutions comprised concepts using eductor systems, standard dredges, Sauerman, dragline and clamshell dredges, offshore breakwaters, jetty extension, weir jetty, updrift traps and enhancement of the ebb currents. The two most feasible long-term solutions were a modification of the 1977-79 multi-year phased dredging program and periodic dredging of a sand trap created by construction of an offshore breakwater. The annual cost of phased dredging was estimated to be $580,000. This solution would be expected to keep the harbor open for eleven months each year as opposed to 8½ months under the current annual dredging program. The offshore breakwater with annual dredging would intially cost $15,500,000 and its annual cost would be $1,510,000. However, this solution would keep the harbor open throughout the year.

ABSTRACT

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SECTION A

THE STUDY AND REPORT

INTRODUCTION

A.1 The small boat harbor at Santa Cruz was constructed in 1962 by dredging an entrance channel and berthing basin and by constructing two jetties through a narrow beach. The jetties and channel immediately began to interupt sand transport, forming a fillet on the west (up-drift) beach, inducing erosion on the east (down-drift) beach, and shoaling the entrance channel. Within a few years, the entrance channel shoaled to the extent that it was effectively closed to navigation for approximately four months each winter. A dredge had to be mobilized to clear the channel each spring. The channel would remain open for navigation through the summer and fall until the severe winter waves accelerated the shoaling. The House Document authorizing construction of the harbor specified that a permanent sand bypassing procedure be developed and turned over to the local authorities.

PURPOSE

A.2 This study was conducted to define and document the littoral processes and to develop a cost effective maintenance or sand bypassing procedure. The study was conducted in 1977 and 1978 in three phases. The purpose of the first phase was to develop an interim multi-year dredging program to maintain the channel until a permanent solution could be found. The purpose of the second phase was to document the littoral processes contributing toward shoaling of the harbor entrance channel. The purpose of the third phase was to develop and evaluate alternate long-term solutions for mitigating the shoaling and keeping the harbor entrance channel open.

SCOPE

A.3 The length of shoreline studied comprised the coastal reach between the San Lorenzo River and Black Point, and the study was limited in scope generally to an analysis of existing data.

A.4 The first phase dredging study was conducted prior to the littoral-processes study so that the interim dredging program could be implemented in the winter of 1977-78. Development of the interim dredging program was based on review of government furnished aerial photography, condition surveys and historical wave data; on review of various sources of coastal data from local universities, U. S. Geological Survey (NOAA), CERC, and WES; and on interviews with WES personnel at Santa Cruz that were conducted to ascertain the results of the Corps' experimental sand bypassing program. Technical information pertinent to the multi-year dredging program included a determination of the length of time harbor closure could be expected or tolerated, the required size or discharge capacity of the dredge or other equipment that might be used in the program, the critical standby periods of the year, the minimum channel dimensions to be maintained during winter, and the best location for sand disposal. This report documents the experiences of the 1977-1978 dredging operation for input in future dredging specifications.

A.5 The littoral processes study was conducted to clearly define the specific local littoral processes that cause shoaling of the Santa Cruz Harbor entrance channel. The study was conducted prior to February 1978 and was updated for this final report. A secondary work item involved an evaluation of existing montoring programs and recommendations for their modification in future applications.

A.6 The third phase study was conducted to develop a minimum of 15 alternative solutions to the maintenance problem that appeared reasonable from the standpoints of cost-effectiveness, of environmental and social acceptability, of plan effectiveness and of engineering practicality. Preliminary plans and cost estimates for the alternative solutions were prepared in sufficient detail for preliminary engineering evaluation. Costs were based on February 1978 price levels.

THE REPORT

A.7 This report comprises six sections. Section A defines the purpose and scope of study and summarizes prior reports. Section B provides a description and history of the harbor and defines the problem. Section C is a detailed analysis of the littoral processes. Section D describes the multi-year interim dredging program, explains its formulation and evaluates its performance in the first year. Section E presents 16 alternative solutions to mitigate the effects of shoaling. Section F presents the conclusions of the study and recommends the most appropriate course of action to pursue, based on evaluation of the impacts and cost of each alternative. References are listed in appendix 1 and energy-flux calculations are contained in appendix 2.

PRIOR REPORTS

A.8 Federal. A report by the San Francisco District, Corps of Engineers, titled "Beach Erosion Control Report on Cooperative Study of Santa Cruz Area, Pacific Coastline of California", dated January 20, 1956, indicated that the possible sources of littoral material for the region are erosion in drainage areas by tributary streams, erosion of sea cliffs, and upcoast littoral transport. An experimental groin was constructed 400 feet west of the present site of the west jetty to determine the littoral transport rate. From June 1955 to April 1956, littoral drift did not accumulate on the west side and a net loss of 7,000 cubic yards was observed on the east side. No determination of the sediment transport rate was made because of the inconclusive results. The study stated that the Twin Lakes Beach shoreline has experienced significant seasonal fluctuations, evidenced by the shoreline reach fronting Woods Lagoon and Schwans Lagoon, which has receded as much as 150 feet during the winter season. The eastern end of Twin Lakes Beach receded to the bluffline during the winter months and aggraded during the summer months. The study concluded from a wave analysis that Twin Lakes Beach shoreline alignment is the result of the groin effect of Black Point.

A.9 A report by the San Francisco District, Corps of Engineers, titled "Survey Report, Santa Cruz Harbor, California", dated May 1957, indicated that three possible sources of littoral material entering Santa Cruz Harbor are offshore deposits, upcoast and downcoast littoral transport of sediments derived from bluff erosion, and coastal streams. The minimum estimated net annual littoral transport rate of 25,000 cubic yards was inferred from the shoaling rate at Moss Landing of 47,000 cubic yards and maintenance dredging at Monterey Harbor of 14,000 cubic yards. An estimated maximum rate of 300,000 cubic yards per year was calculated from analyses of sediment discharge rates of streams between Halfmoon Bay and Santa Cruz and from bluff erosion rates between Point Ano Nuevo and the San Lorenzo River. The study concluded that the jetties proposed by the survey report would act as littoral barriers which would trap sand, stabilizing the shoreline between Woods Lagoon and the San Lorenzo River. Downcoast erosion was projected to be alleviated by placing materials dredged from Woods Lagoon during construction on the downdrift beach to the east. Two methods of coping with the shoaling were to construct an offshore breakwater to impound sand and intermittently bypass the entrapped sand, or to bypass littoral drift by stationing a floating dredge in the entrance channel.

A.10 House Document No. 357, 85th Congress, 2nd Session, July 3, 1958, authorized the construction of a light-draft harbor in Woods Lagoon at the eastern boundary of Santa Cruz. The authorized project consisted of an entrance channel, an inner channel, a turning basin, two rubblemound jetties, and a sand-bypassing plant.

A.11 A report by the San Francisco District, Corps of Engineers, titled "Santa Cruz Harbor, California, Design Memorandum No. 1," dated December 1960, designed the improvements of Santa Cruz Harbor as described in House Document No. 357, 85th Congress, 2nd Session. The designs for project structures were based on a design wave with a height of 21 feet and a period of 16 seconds. The report stated that if the actual littoral transport rate approached the minimum value of 25,000 cubic yards per year, only normal maintenance dredging would be required. If the actual rate approached the maximum value of 300,000 cubic yards per year, then a floating sand-bypassing or maintenance plant would be provided in accordance with the project document.

A.12 A report submitted by V. J. Grauzinis, Oceanographic Consultant from La Jolla, California, titled "An Analysis of Seiche Conditions in Santa Cruz Harbor, California, and Some Implications for the Proposed Harbor Extension", prepared for the San Francisco District, Corps of Engineers, under Contract No. DACW07-68-C-0034, dated March 1968, reported the findings of a study on long-wave excitation in the existing harbor and predicted that seiche conditions would occur in the new harbor complex. One of the essential findings of the study was that the high seiches occurring in the existing harbor are the result of the high levels of excitation in the adjoining portion of Monterey Bay and are not due to sharp resonance amplification. The investigation also determined that the proposed configuration of the harbor extension would reduce the seiche amplitudes from those present in the existing basin.

A.13 A report by the San Francisco District, Corps of Engineers, titled "Detailed Project Report, City of Capitola, Beach Erosion Study, Santa Cruz County, California", dated December 1969, stated that the impoundment of littoral drift by the west jetty after its construction and the protection afforded to the bluffs along West Cliff Drive by the accreted beach may have caused some of the erosion in Capitola. However, the study concluded that the main source of littoral material at Capitola was sediment discharge from Soquel Creek.

A.14 A report by the San Francisco District, titled "Detailed Project Report, Extension of Existing Small Craft Harbor, Santa Cruz, California", Section 107 Study, dated October 1970, recommended the extension of the existing small-craft harbor into Woods Lagoon. The report justified the proposed extension by the demand for additional slips and by an analysis which indicated that the project was economically sound. The benefit-to-cost ratio was estimated to be 4.0 to 1.0. The proposed extension would add 455 slips and approximately 34 acres to the existing small-craft harbor.

A.15 A report by the San Francisco District, Corps of Engineers, titled "Office Report, Santa Cruz Harbor Sand Bypassing Plant, Santa Cruz County, California", dated August 1971, reported results of an investigation of the various methods of sand bypassing considered for Santa Cruz Harbor. These methods

included: a rail-mounted dredge located on the west jetty; cableway buckets; a fixed suction dredge with a submerged pipeline; and a 12-inch hydraulic pipeline dredge. The design capacity of the proposed bypassing plant was 300,000 cubic yards per year; this capacity was determined from the rate of sand impounded by the west jetty after its construction. The report recommended a hydraulic dredge for the Santa Cruz Harbor maintenance and bypassing operations. The proposed maintenance dredging schedule stipulated that dredging operations would normally not be performed between 15 May and 15 October. A sediment storage basin would be created for the storm season by dredging the channel to full project depths plus overdepth in November or December. The maintenance dredging schedule also stipulated that the channel would be dredged to the project dimensions at the end of the storm season in April or May to provide a navigable channel during the summer months.

A.16 A report by the San Francisco District, Corps of Engineers, titled "Prototype Stability Study Of Quadripod Armor Units At West Jetty, Santa Cruz Harbor, Santa Cruz, California", dated August 1971, attempted to verify experimentally determined stability coefficients under prototype conditions. The movements of the quadripods placed on the west jetty and the incident wave heights were recorded. Conclusive verification was not possible because the incident wave heights were never larger than 13.6 feet, which is 65 percent of the design wave height of 21 feet.

A.17 A report by the San Francisco District, Corps of Engineers, titled "Santa Cruz Harbor Office Report -- Sand Bypassing Plant", dated February 13, 1974, evaluated possible sand-bypassing systems and other methods of maintaining a navigable channel for the entire year. Analysis of hydrographic surveys from March 1973 to January 1974 revealed that shoaling of the entrance channel begins at the head of the west jetty and advances to the head of the east jetty. Review of earlier hydrographic surveys indicated that shoaling of the channel continues and migrates farther into the channel with successive winter storms. On the basis of a site inspection of the west jetty, leakage of sand through the jetty was estimated to be less than ten percent. The report concluded that jet-pump bypassing would be the most practical and economical solution to the shoaling problem, and that sealing the voids in the jetty should be deferred until after a bypass system has been installed.

A.18 A report by the San Francisco District, Corps of Engineers, titled "Environmental Statement, Draft, Maintenance Dredging (FY1974), Santa Cruz Harbor, Santa Cruz, California", dated March 1974, identified the following processes, listed in order of magnitude, as the causes for the shoaling of the harbor: (1) bypassing of sand around the head of the west jetty; (2) leakage of sand through the porous west jetty; (3) aeolian transport of sand over the west jetty; and (4) transport of sand around the east jetty during littoral transport reversals.

A.19 A report by Moffatt & Nichol, Engineers, titled "Interim Dredging Program for Santa Cruz Harbor, California", prepared for the San Francisco District, Corps of Engineers, under Contract No. DACW07-77-C-0023, dated November 1977, described a plan for a multi-year dredging program aimed at maintaining the channel open year-round until a permanent solution to the shoaling problem is implemented. The study recommended periodic dredging in the entrance channel during the winter with a dredge stationed in the harbor from November through May.

A.20 A report by Moffatt & Nichol, Engineers, titled "Littoral Processes Study for Santa Cruz Harbor, California", prepared for San Francisco District, Corps of Engineers, under Contract No. DACW07-77-C-0023, dated February 1978, described the littoral processes and estimated the quantities of sand shoaling the harbor entrance channel. A net littoral transport rate off the entrance channel of 300,000 cubic yards per year was estimated. An estimated 100,000 cubic yards per year were forming a tip shoal in the entrance channel, 20,000 cubic yards were leaking through the voids in the west jetty, and 1000 cubic yards per year were leaking through the east jetty. An estimated shoaling of 20,000 cubic yards per year was attributed to reversal in transport, including the effects of rip current and offshore-onshore transport at the head of the east jetty. Wind transport and onshore transport from the offshore region were estimated to be 7000 and 2000 cubic yards per year respectively. Sand was estimated to be bypassing the entrance channel at a rate of 175,000 cubic yards per year. Annual dredging, using the dredging practices prior to 1977 winter, was estimated at 150,000 cubic yards per year.

A.21 A report by Moffatt & Nichol, Engineers, titled "Feasibility Study to Mitigate Shoaling in Santa Cruz Entrance Channel", prepared for San Francisco District, Corps of Engineers, under Contract No. DACW07-77-C-0073, dated March 1978, presented and evaluated 16 alternative solutions to the shoaling problem. The alternatives were classified as maintenance, bypassing, and structural solutions. Feasibility of each solution was determined based on cost-effectiveness, reliability, and environmental impact. The report was prepared for evaluation by the Federal Tidal Hydraulics Committee.

A.22 Others. A technical report by T. E. Yancey of the Hydraulic Engineering Laboratory, at the University of California, Berkeley, California, titled "Recent Sediments of Monterey Bay, California", dated July 1969, reported that different sediment types in Monterey were found in three widespread bands which generally were aligned parallel to the submarine contours of Monterey Bay. The outermost band which predominately consists of coarse grained sediments with some cobbles occurs along the seaward edge of the continental shelf and ranges in width from one to three miles. The middle band which consists of very fine grained muds is approximately three to four miles wide and was found in water depth between 150 and 300 feet. The innermost band along the present shoreline comprises coarse and medium grained sediments that are being worked by waves close to shore. The study reported that the cover sediments were not simply reworked in place, but that sediment is being moved into deeper water from the shoreline, and being sorted by size, with the finest sediments being carried farthest from the shoreline. The report identified three major sediment sources for Monterey Bay. The two largest are the Salinas and Pajaro rivers and the remaining source is downcoast transport of beach sand in the northern portion of the Bay. San Lorenzo River was not determined to be a significant source of sediments. Investigation of mineral provinces indicated that most sediments remained in the local area, with longshore transport distributing the coarser sediments in the longshore direction. Grain-size distributions indicated that large quantities of sediments were moving directly offshore from the beaches.

A.23 A report prepared by Earl and Wright, Consulting Engineers, San Francisco, California, titled "Dredging Report On Santa Cruz Harbor, Santa Cruz, California", for the Santa Cruz Port District, dated June 1, 1970, reported on an investigation of the shoaling problem of Santa Cruz Harbor. The report identified the movement of sand from west to east along the Santa Cruz beaches as being the primary cause of the shoaling. From observations of updrift beach erosion and from sediment discharge volumes of the San Lorenzo River, it was estimated that the total volume of littoral drift was approximately 300,000 cubic yards per year. The report estimated that a bypassing system should be capable of handling 175,000 cubic yards per year. The remainder was assumed to naturally bypass the channel mouth. The recommendation with regard to the alleviation of the shoaling problem was to acquire a 14-inch floating dredge which would perform the necessary sand-bypassing and maintenance dredging.

A.24 A Master's thesis by Richard G. Anderson, United States Naval Postgraduate School, titled "Sand Budget For Capitola Beach, California", dated March 1971, determined from monthly wave energy-flux calculations that the net littoral transport for Capitola was downcoast for the entire year. The major source of sediment in the study area was found to be the upcoast beaches at Santa Cruz. Annual potential drift rates of 352,000 and 310,000 cubic yards per year were determined for the Santa Cruz and Capitola shorelines, respectively. The study concluded that the depletion of sand from Capitola Beach in 1965 was the result of construction of Santa Cruz Harbor.

A.25 A report by Jon T. Moore of the University of California at Berkeley, titled "A Case History of Santa Cruz Harbor, California", dated June 1972, reviewed the Santa Cruz Harbor Project and maintenance problems since its construction. The study described the shoreline between the San Lorenzo River and Soquel Creek prior to the construction of the harbor as being a site of active erosion. From a comparison of profiles measured along the centerline of the channel, the study determined that shoaling of the channel starts at about the dogleg in the entrance channel. Waves diffracting around the head of the west jetty appear to transport material farther inside the channel. Calculations of upcoast drift bypassing around the east jetty by waves

from the southeast to south-southeast indicated that shoaling during reversals in littoral transport was not significant.

A.26 A report by Dr. Douglas L. Inman, Scripps Institution of Oceanography, titled "Summary Report Of Man's Impact On The California Coastal Zone", for the California Department of Navigation and Ocean Development, dated November 1976, stated that Santa Cruz is situated near the southern terminus of a littoral cell that begins at the mouth of San Francisco Bay. The report concludes that of the total sediment supply to the Santa Cruz area is 200,000 cubic yards of littoral drift per year from upcoast sources and an additional 100,000 cubic yards per year by sediment discharge from the San Lorenzo River. The report stated that the shoaling process appears to involve the removal and deposition of beach sand offshore by short-period, steep storm waves from the southwest; subsequent redistribution of the sand by long-period swells from the west moves the drift shoreward back onto the beach and into the channel. A number of remedial actions were suggested, including overdredging of the entrance channel to provide storage for the littoral drift, the creation of a sand trap seaward of the west jetty either by overdredging or by the construction of a detached breakwater, and the installation of a crater-sink, fixed sand-bypassing plant at the harbor entrance.

INVOLVED PARTIES

A.27 In January of 1951, the Santa Cruz Port District became a legally constituted public body. The goal of the District was to improve the waterways and harbor facilities. The design of the inner harbor, including bulkheads, revetments, berthing facilities, and public landing, was performed by an engineering firm retained by the Port District. The operation of the harbor since its construction has been the responsibility of the District.

A.28 The participation of the Waterways Experiment Station in the Santa Cruz Harbor shoaling problem involved the installation of an experimental jet-pump, fixed sand-bypassing plant on the west jetty. The project was authorized in 1975 as a one-year pilot operation to test the capability, cost, and reliability of the jet-pump system. The system began operations in July 1976 and tests were completed in March 1978.

SECTION B

THE PROBLEM AND ITS CAUSES

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SECTION B

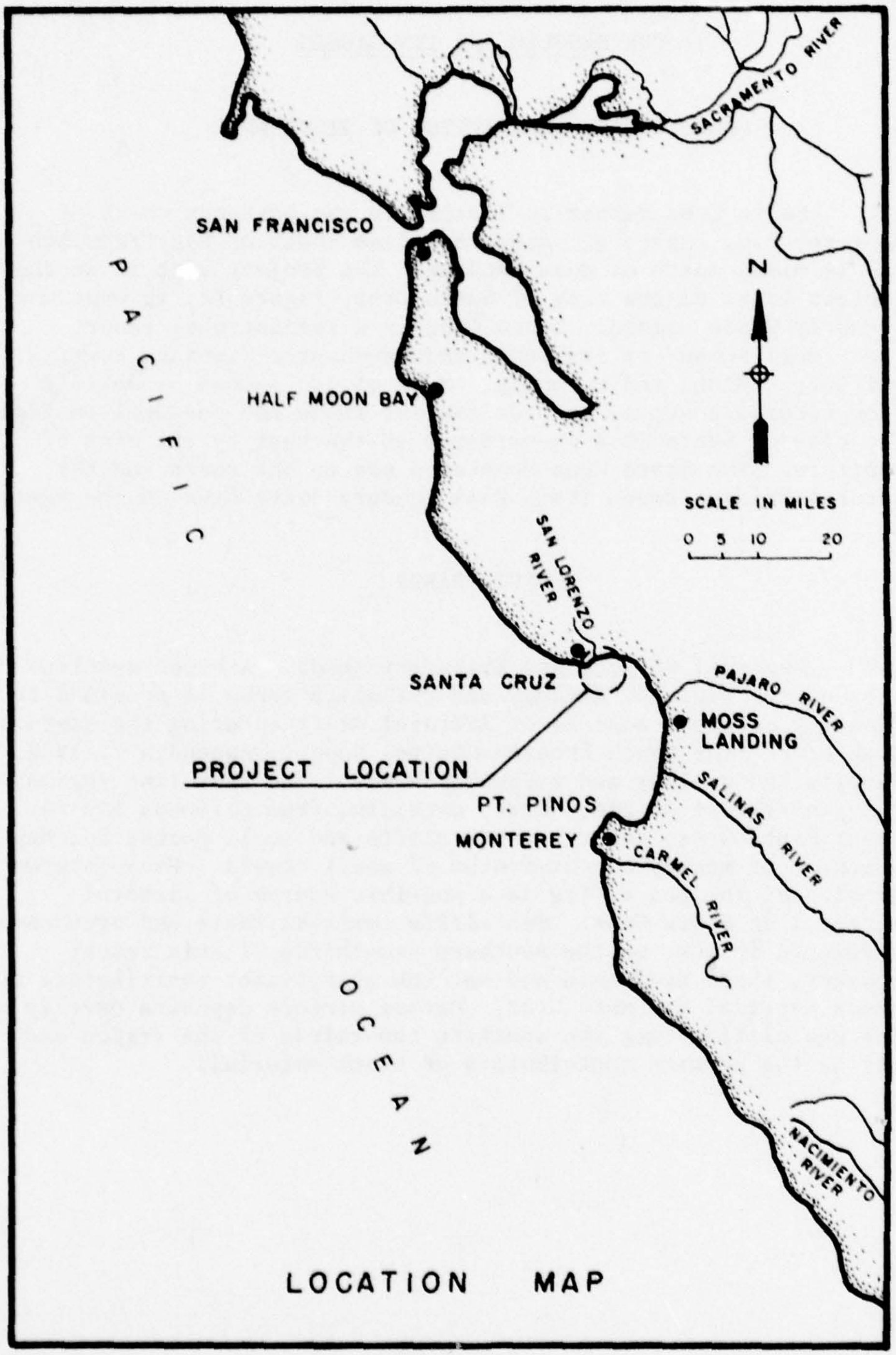
THE PROBLEM AND ITS CAUSES

LOCATION AND DESCRIPTION OF STUDY AREA

B.1 Santa Cruz Harbor is located on the northern coast of Monterey Bay, figure B1, about 65 miles south of San Francisco and 14 miles north of Moss Landing. The project site is at the eastern limit of the city of Santa Cruz, figure B2, in what was formerly Woods Lagoon. Santa Cruz is a recreational resort area, well known for its ocean and freshwater fishing, boating, golfing, riding, and swimming. Much of its income is derived from outside sources, such as tourist trade and seasonal residents. The city of Santa Cruz is bordered on the east by the city of Capitola. The Santa Cruz Mountains are to the north and the Natural Bridges Beach State Park borders Santa Cruz on the west.

PHYSIOGRAPHY

B.2 Regional Geology and Tributary Areas. A brief description of the regional geology and tributary areas is provided to identify possible sources of littoral drift entering the Santa Cruz area. The Beach Erosion Control Report, Appendix 2, 1956, details the geology and tributary areas. The shoreline region lying northwest of Santa Cruz, extending from Halfmoon Bay to Point Santa Cruz, comprises sea cliffs and small pocket beaches which occur mostly at the mouths of small creeks. Wave-induced erosion of the sea cliffs is a possible source of littoral material at Santa Cruz. Sea cliffs comprise shale and cretaceous sediments in most of the southern two-thirds of this reach; however, these sediments are not the significant contributors of beach material at Santa Cruz. Marine terrace deposits overlay the sea cliffs along the southern two-thirds of the region and may be the primary contributors of beach material.



SACRAMENTO RIVER
SAN FRANCISCO
PACIFIC
HALF MOON BAY
N
SCALE IN MILES
0 5 10 20
SAN LORENZO RIVER
SANTA CRUZ
PAJARO RIVER
MOSS LANDING
PROJECT LOCATION
SALINAS RIVER
PT. PINOS
MONTEREY
CARMEL RIVER
OCEAN
NACIMIENTO RIVER
LOCATION MAP

FIGURE B1.

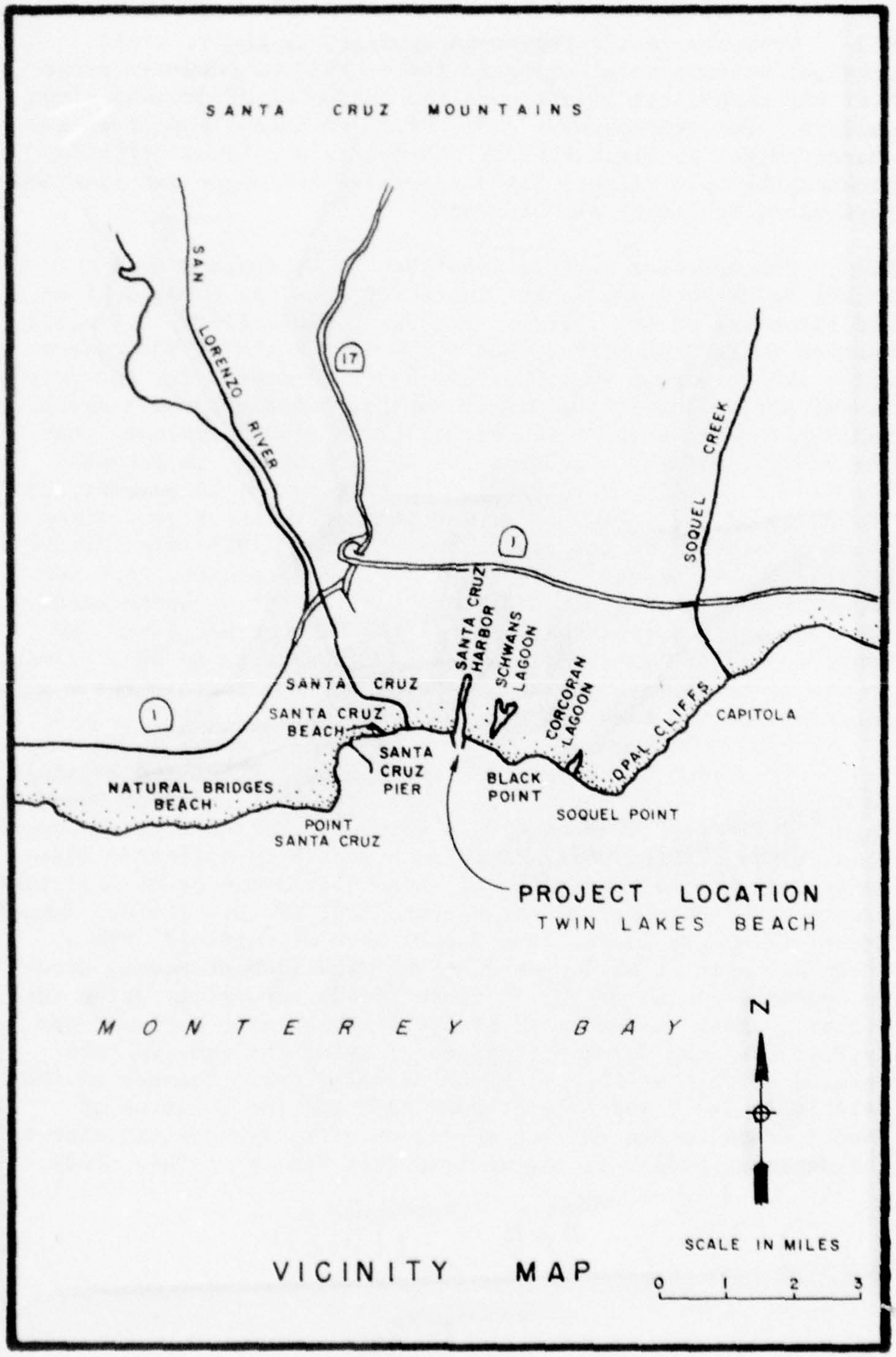

SANTA CRUZ MOUNTAINS
SAN LORENZO RIVER
17
SOQUEL CREEK
1
SANTA CRUZ HARBOR
SCHWANS LAGOON
CORCORAN LAGOON
SANTA CRUZ
SANTA CRUZ BEACH
1
OPAL CLIFFS
CAPITOLA
SANTA CRUZ PIER
BLACK POINT
NATURAL BRIDGES BEACH
POINT SANTA CRUZ
SOQUEL POINT
PROJECT LOCATION
TWIN LAKES BEACH
MONTEREY BAY
N
VICINITY MAP
SCALE IN MILES
0 1 2 3

FIGURE B2.

B.3 Drainage in the region is typified by short, steep-gradient streams originating in the Santa Cruz mountain range. Near the ocean, the rivers have the character of drowned river valleys. The streams have small drainage areas, and flows are characterized by flash floods. No deltas are found offshore because the wave climate rapidly removes sediments and distributes them along the beach and offshore.

B.4 The reach of shoreline between Point Santa Cruz and Soquel Point includes Santa Cruz Beach, the San Lorenzo River, and stretches of sea cliffs. The San Lorenzo River, 0.5 miles west of Santa Cruz Harbor, has a drainage basin of 137 square miles and an annual runoff of 125,000 acre-feet. The heaviest runoff occurs during the winter months, October through April. Estimates made by the Corps of Engineers (1974) indicate that the annual sediment discharge of the San Lorenzo is between 88,000 and 133,000 cubic yards. Of this total, 20 percent, or 18,000 to 27,000 cubic yards, was assumed to be of sand sizes found on beaches in the study area. Yancey (1968) also determined, by studying minerals of the river basin and beaches, that the San Lorenzo River is not a primary contributor of beach sand. The sea cliffs immediately east of the San Lorenzo River are Monterey shale. These cliffs are not considered to be a primary source of beach material because they neither contain beach size material nor are they exposed to severe wave attack.

B.5 The reach of shoreline between Soquel Point and Capitola is characterized by sea cliffs. The sea cliffs east of Soquel Point, Opal Cliffs, are actively eroding. Marine terrace deposits blanket the cliffs, which comprise a poorly consolidated clay-to-gravel material. Erosion of these cliffs may be an important source of beach material for beaches east of the cliffs. Soquel Creek, which discharges into Soquel Cove at Capitola, has a drainage basin of 42.4 square miles. The flow of Soquel Creek is ephemeral in nature, with flash floods occurring during the winter season. A sand load of 8,000 cubic yards per year was estimated by the Corps of Engineers, using the same factors assumed in the San Lorenzo River calculations. Because of the relatively small quantity of sand load and the location of Soquel Creek to the east of the study site, its contribution to the sediment budget is not an important factor in this study.

B.6 Offshore of the project site is a narrow continental shelf which drops off into the Monterey Canyon located in the middle of Monterey Bay. Figure B3 shows the general bathymetric features of the Santa Cruz embayment. The beach slopes steeply to the 20-foot depth contour, then slopes very gently to the south. The bottom comprises hard rock outcroppings which are partially overlain with unconsolidated sediment.

B.7 Shoreline Reaches. For purposes of discussion, the study shoreline was divided into three shoreline reaches: upcoast, Twin Lakes Beach, and downcoast. The upcoast reach extends from Point Santa Cruz to the San Lorenzo River. Point Santa Cruz, which is near the northwestern limit of Monterey Bay, shelters Santa Cruz Harbor from ocean swell arriving from the north and northwest. The emphasis of the study was concentrated on the Twin Lakes Beach reach, a highly developed recreational beach fronting Santa Cruz Harbor. Twin Lakes Beach is the reach that extends from the San Lorenzo River eastward to a headland known as Black Point. This headland acts as a natural groin which stabilizes the Twin Lakes Beach shoreline. The downcoast shoreline reach extends from Black Point to the city of Capitola. Soquel Point, which is the western terminus of the Opal Cliffs, is located in the downcoast reach.

B.8 The upcoast reach of the study area contains two distinctly different shoreline segments, a north-south segment and the Santa Cruz Beach segment. The north-south segment consists of sea cliffs about 25 feet in height fronted by a wave-cut terrace. The southern terminus of this segment is Point Santa Cruz, and the northern terminus is Santa Cruz Beach. The Santa Cruz Beach segment, a highly developed recreational beach, is aligned in an east-west direction. This section of the shoreline is not backed by bluffs.

B.9 The Twin Lakes Beach reach can be further divided for discussion into two segments. The western or updrift segment is bounded by the San Lorenzo River to the west and by Santa Cruz Harbor to the east. At the western terminus of this reach, immediately east of the mouth of the San Lorenzo River, a natural rock groin projects from the shoreline. The updrift shoreline is characterized by low bluffs which are fronted by

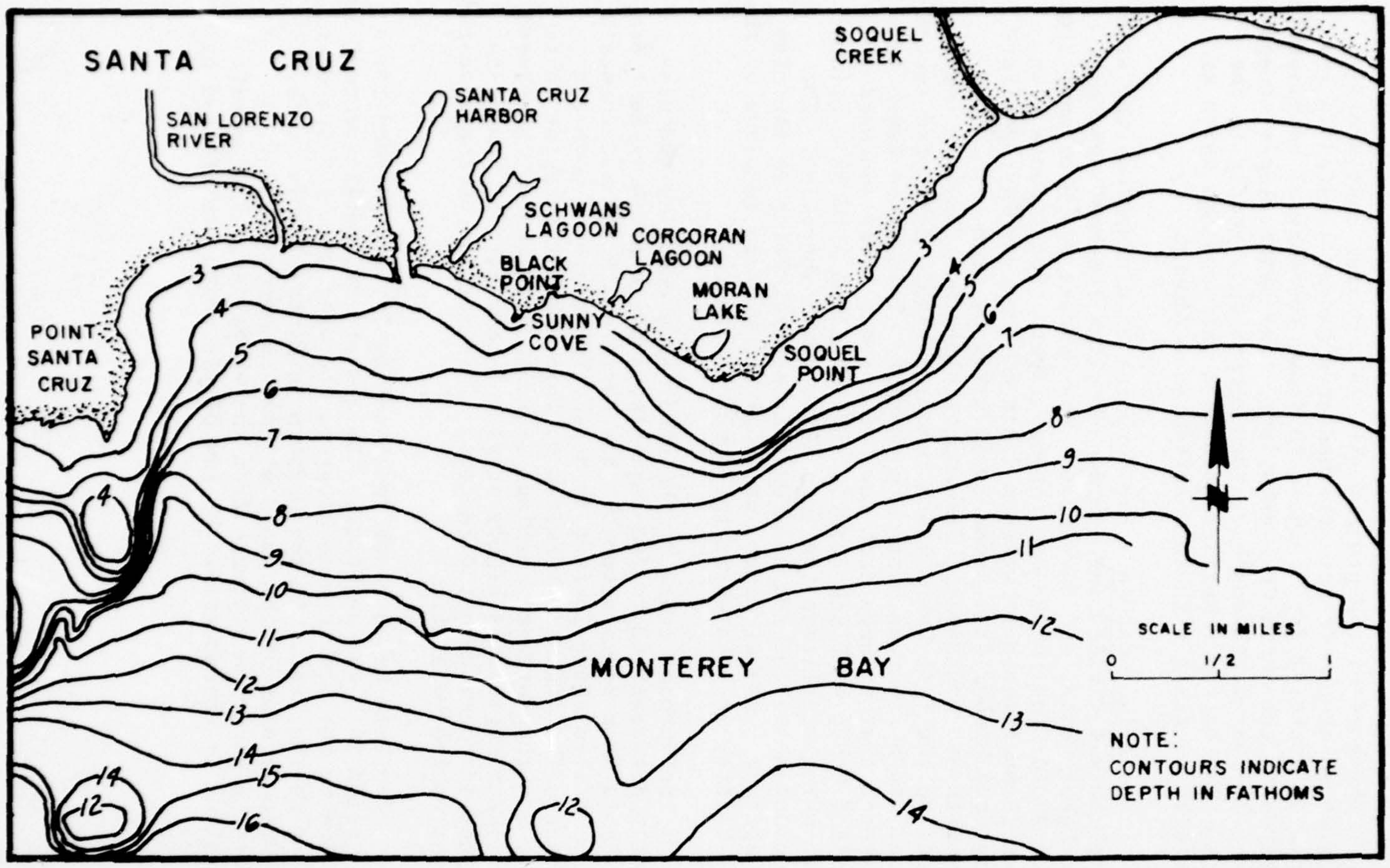


FIGURE B3. BATHYMETRIC FEATURES IN SANTA CRUZ REGION

a wide sandy beach. Prior to the construction of the Harbor jetties, the beach did not protect the bluff from wave attack at high tide. The eastern or downdrift segment extends from the harbor entrance to Black Point. The western half of this segment, which fronts the harbor and Schwans Lagoon, consists of a sand beach approximately 2000 feet long. The eastern half of the downdrift segment is backed for most of the distance by bluffs about 35 feet in height.

B.10 The western and eastern boundaries of the downcoast reach are Black Point and the city of Capitola, respectively. A second prominent headland, Soquel Point, is located one mile east of Black Point. The shoreline between Black Point and Sunny Cove east of Black Point is characterized by rocky ledges which are resistant to wave attack. From Sunny Cove to Soquel Point, the shoreline consists of bluffs which average 35 feet in height. The bluff line is broken at two locations by Corcoran Lagoon and Moran Lake. The shoreline from Soquel Point to Capitola is about 14,000 feet long and is backed by the Opal Cliffs, which range in height from 35 to 75 feet. For most of the year, the Opal Cliffs shoreline is devoid of beach material, which leaves the cliffs exposed to wave erosion.

B.11 Beach Material. The size distribution of beach materials determined from the Littoral Environment Observation (LEO) program are given in table B1. Data were obtained from sand samples taken in the splash zone for various months in 1970 and 1971 at Twin Lakes Beach. Sediment samples were also taken at four stations shown on figure B4 in the Santa Cruz Harbor channel by the U.S. Army Corps of Engineers in May 1971 and July 1973. Size distributions for these sediments, obtained by a mechanical analysis, are shown in the gradation curves in figure B5. Size distributions for these sediments are also given in table B1. Sand sizes were medium-to-fine, with sediments in the channel being slightly finer than the sand found in the splash zone.

B.12 Shoreline Alignment. The general east-west orientation of Twin Lakes Beach reach shoreline is maintained at this location by Black Point, a natural rocky headland 2,500 feet west of the east jetty. The August 1977 shoreline azimuths of the updrift and downdrift beaches were 285° and 292°, respectively. The shoreline azimuth is defined herein as the angle measured

Table B1.
SEDIMENT ANALYSIS STATISTICS

Source: LEO Program, Twin Lakes, California (36 57.70N 121 59.76 W)

Sample Date	Phi* Mean Dia.	Phi Std. Dev.	Phi Skew-ness	Phi Kur-tosis	Median Dia. (mm)
2-28-70	.75	.46	.93	3.43	.65
3-17-70	.81	.68	.22	2.66	.60
4-18-70	1.52	.40	.33	2.64	.35
5-4-70	1.85	.38	.32	1.97	.29
6-1-70	1.77	.45	.10	2.42	.29
6-1-70	1.89	.40	.42	2.56	.28
7-4-70	2.10	.35	.52	2.79	.24
11-3-70	1.60	.43	1.33	5.30	.35
11-17-70	1.82	.37	.66	2.86	.30
12-4-70	1.49	.53	.79	3.66	.37
10-31-71	1.95	.38	.65	4.13	.26
12-2-71	1.55	.36	.19	2.34	.34

Source: U.S. Army Corps of Engineers

Sample Date	Sta.	Phi Mean Dia.	Phi Std. Dev.	Median Dia. (mm)
March 1971	B	2.02	.42	.26
March 1971	D	2.75	.57	.15
July 1973	SC-1	2.16	.91	.26
July 1973	SC-2	1.95	.66	.27

* Wentworth Soil Classification:

			SAND				
Granular	Very Coarse	Coarse	Medium	Fine	Very Fine		
-2	-1	0	1	2	3	4	Phi units
4	2	1	.5	.25	.125		mm

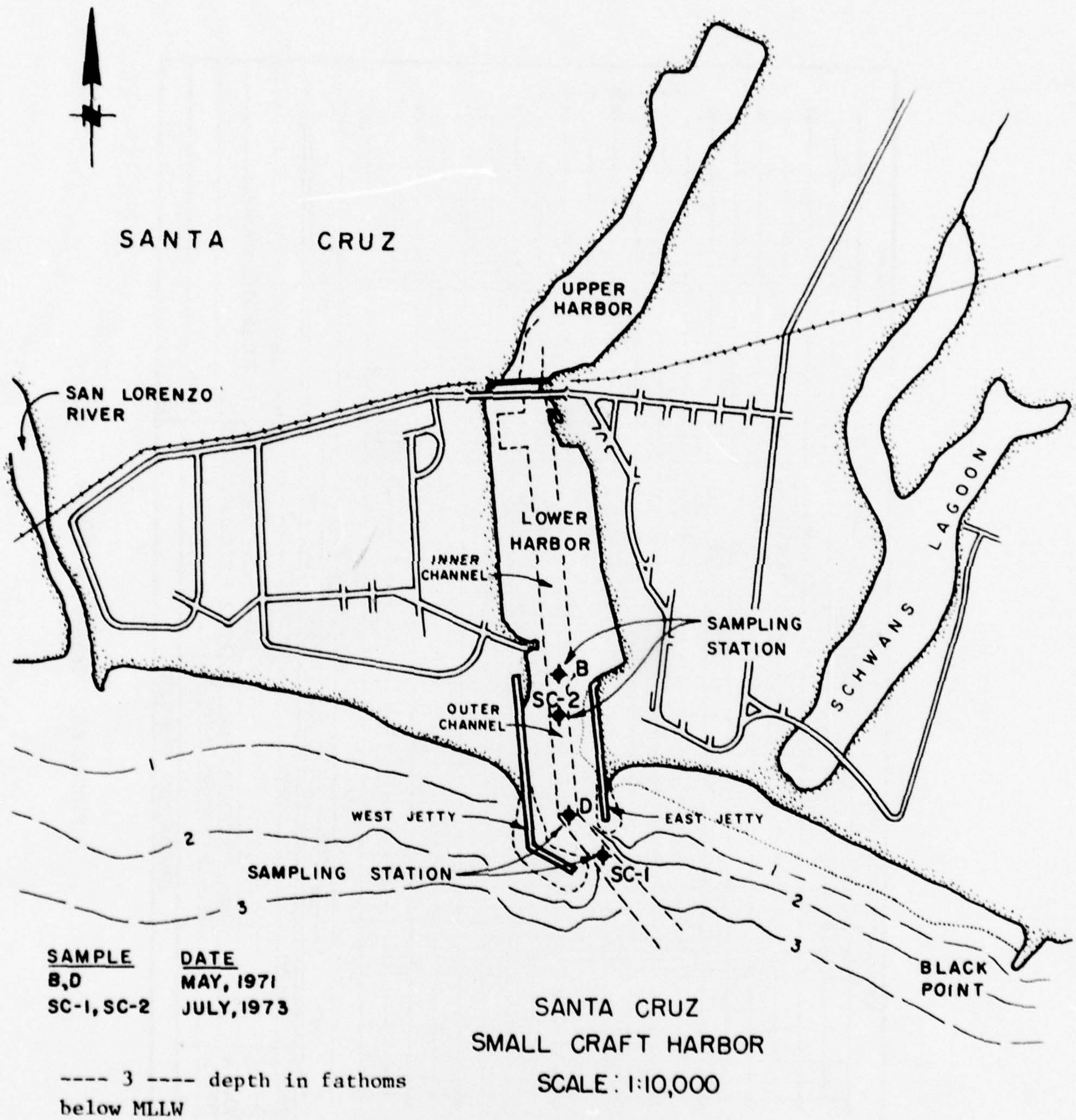


FIGURE B4. LOCATION OF SEDIMENT SAMPLES

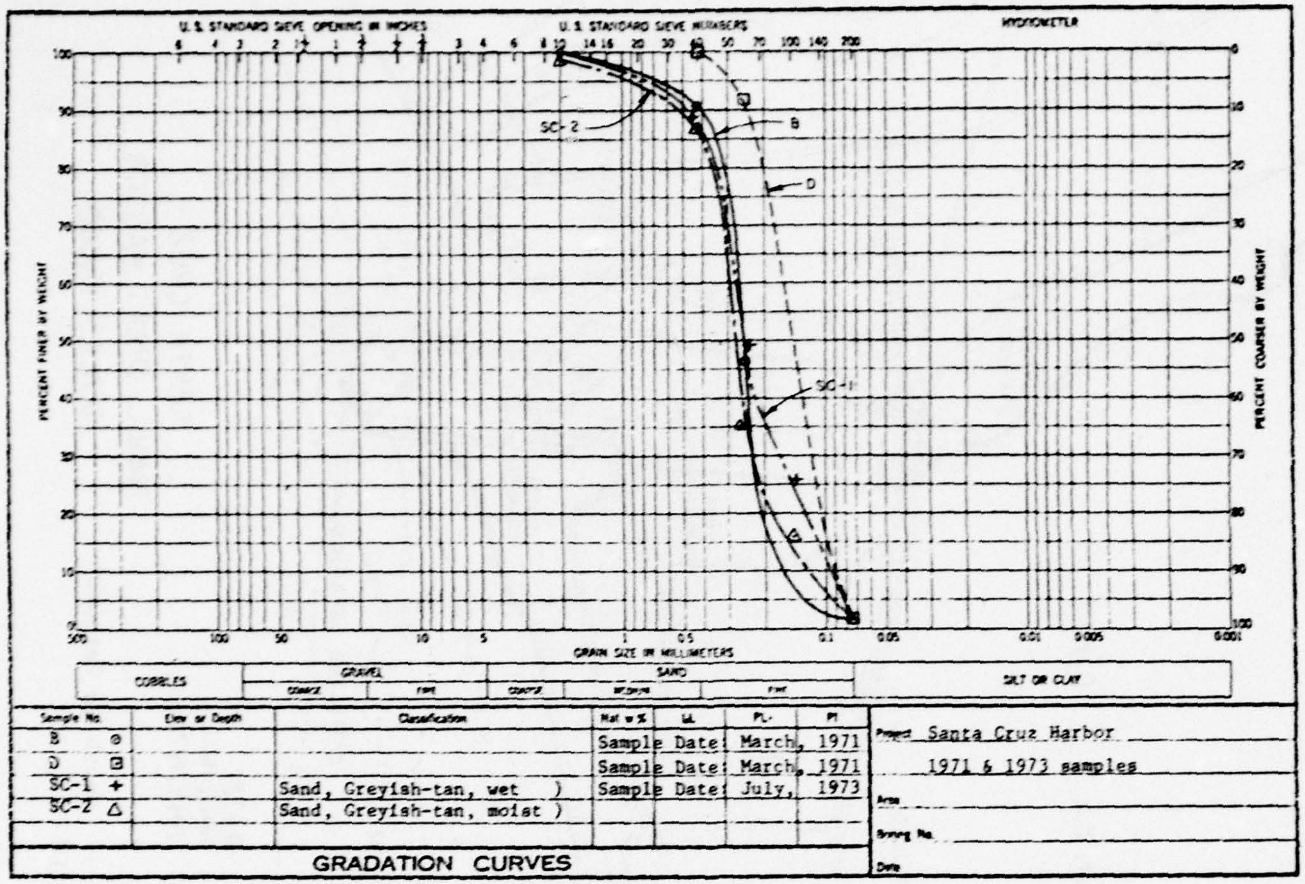


FIGURE B5. Gradation Curves

clockwise from true north to the general trend of the shoreline in the upcoast direction. The small difference between the azimuths of the updrift and downdrift segments indicates that the shoreline has established equilibrium in response to the wave climate and sediment supply in each segment. The orientation of the shoreline is an important factor in defining the shore processes.

SHORE AND HARBOR HISTORY

B.13 The earliest hydrographic surveys of the Santa Cruz region were performed by the United States Coast and Geodetic Survey (U.S.C. & G.S.) during the period from 1851 to 1860. The area was resurveyed by the U.S.C. & G.S. in 1932. A Beach Erosion Control Report of Santa Cruz beach by the Corps of Engineers in 1956 determined by comparison of the surveys that shoreline and offshore depths from the San Lorenzo River to a point one mile east of Soquel Creek had experienced significant erosion. Comparison of the offshore depth changes revealed that a general deepening of the original Santa Cruz Harbor near the municipal pier, one mile west of the Harbor, had occurred during the seventy-two-year period between 1860 and 1932.

B.14 Reports of shoreline conditions for the years preceding the construction of the harbor indicate that the bluffs backing the western segment of Twin Lakes Beach had experienced significant recession. The bluffs, fronted by a narrow beach, were exposed to wave attack at high tide and were eroded an average of two feet per year from 1907 to 1958. The beach immediately east of the harbor fronting Woods Lagoon and Schwans Lagoon was 200 feet wide off East Cliff Drive in the summer. During the winter months, the beach receded to a width of only 50 feet. A sandy beach formed in front of the bluffs at the eastern terminus of Twin Lakes Beach during the summer months. Erosion of the bluffs occurred in the winter months when the high-water line receded to the foot of the bluffs.

B.15 Construction of Santa Cruz Harbor was authorized by Congress under the River and Harbor Act of 1958 by House Document No. 357, 85th Congress, 2nd Session, which provided for a

harbor for light-draft vessels in Woods Lagoon at the eastern boundary of Santa Cruz. The authorized improvements included two rubblemound jetties 1200 feet long and 810 feet long, on the west and east sides of an entrance channel, respectively; an entrance channel approximately 900 feet long, 100 feet wide and 20 feet deep, reducing to 15 feet in depth at the same width for an additional 370 feet; an inner channel, 800 feet long, 150 feet wide, 15 feet deep, reducing to 10 feet in depth at the same width for an additional 600 feet; a turning basin approximately 300 feet long, 250 feet wide, and 10 feet deep; and a sand-bypassing plant. Figure B6 shows project features. The armor units of the seaward side of the west jetty are 28-ton quadripods. The west jetty also has a concrete cap extending to elevation +16 feet MLLW.

B.16 Construction of the harbor was initiated in February of 1962 with the dredging of Woods Lagoon and the start of work on the west jetty. The west jetty was completed in February, 1963; then work began on the east jetty, and it was completed in April 1963. The entrance channel was dredged to the project dimensions in the summer of 1963. Construction of Santa Cruz Harbor was completed in November 1963, with the exception of the sand-bypassing plant. Construction of the sand-bypassing plant was deferred until the littoral drift rate could be more accurately determined.

B.17 During and subsequent to construction of Santa Cruz Harbor jetties, the beach west of the jetties experienced rapid accretion, building out from the 1962 pre-project shoreline about 250 feet by 1965 and about 500 feet by 1977. The growth of the beaches has protected the bluffs along East Cliff Drive from rapid erosion by wave action. The beach east of the jetties soon receded 150 feet and the beach at Capitola vanished.

B.18 Shoaling of the channel has required annual maintenance dredging reported to be on the order of 100,000 cubic yards per year since 1965. In accordance with the authorizing document, a 12-inch hydraulic dredge was delivered to the local interests in 1972 to maintain the channel. However, the dredge was not sufficiently seaworthy to maintain the channel during the

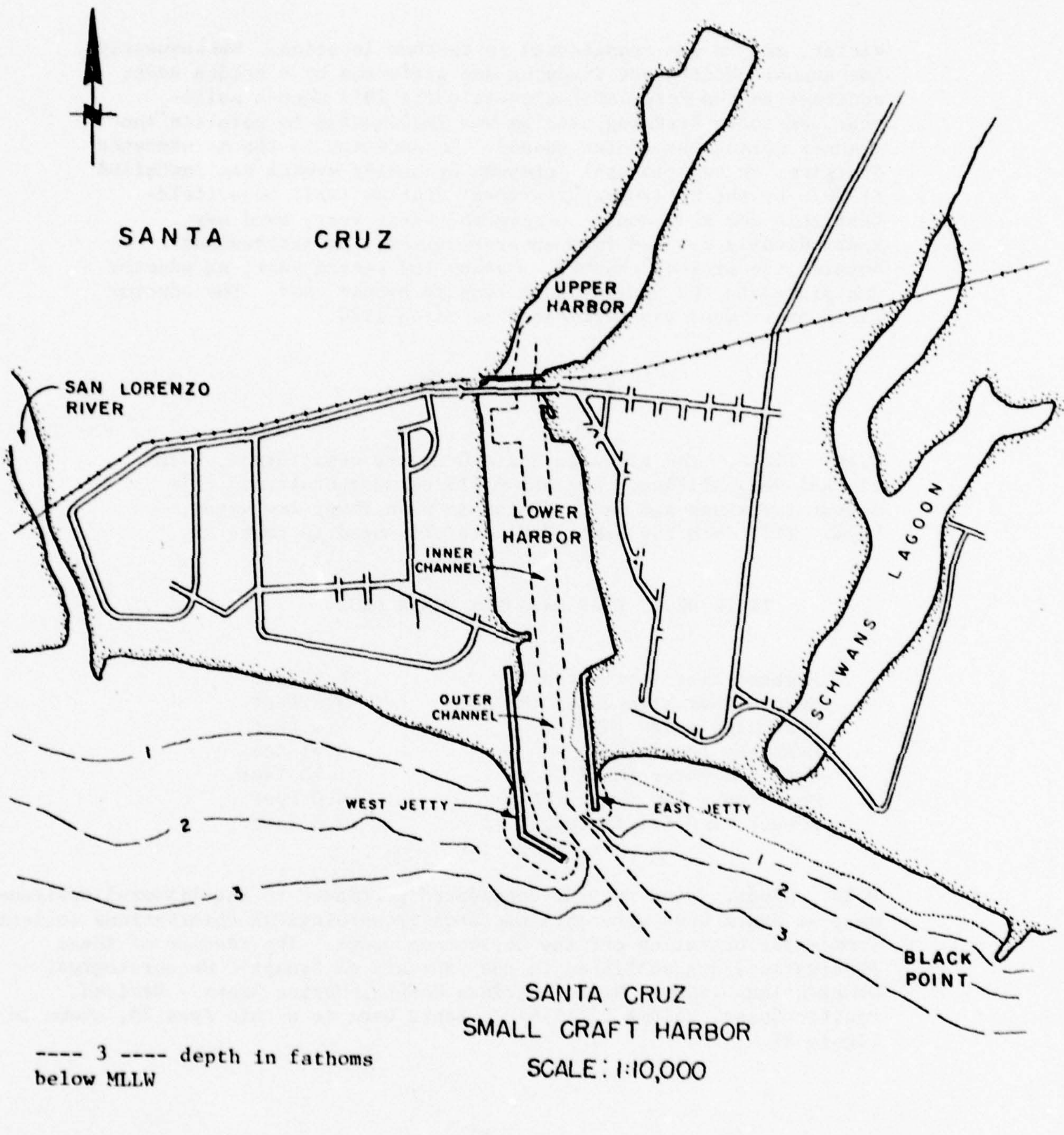


FIGURE B6. PROJECT FEATURES

winter, and it was transferred to another location. Subsequently the annual maintenance dredging was performed by a dredge under contract to the Corps of Engineers until 1977 when a multi-year, periodic dredging program was implemented to maintain the channel during the winter season. In addition to the maintenance dredging, an experimental jet-pump bypassing system was installed in 1976 by the Waterways Experiment Station (WES) to a field-test this new equipment. During the first year, sand was aperiodically removed from an area between the jetties but outside the project channel. During the second year, an eductor was placed in the updrift surf zone to bypass sand. The eductor field experiment was terminated in March 1978.

NATURAL PHENOMENA

B.19 Tides. The tides in Santa Cruz are semidiurnal, with diurnal inequalities. The datum plane incorporated in this report for tides and water depths is mean lower low water, MLLW. Tide data for Santa Cruz are presented in table B2.

Table B2. TIDE DATA FOR SANTA CRUZ

Highest tide (estimated)	8.0 feet
Mean higher high water (MHHW)	5.3 feet
Mean high water (MHW)	4.6 feet
Mean sea level (MSL)	2.91 feet
Mean low water (MLW)	1.10 feet
Mean lower low water (MLLW)	0.0 feet
Lowest tide (estimated)	-2.5 feet

B.20 Winds. Wind records considered pertinent to the littoral environment at Santa Cruz were obtained from meteorological observations collected from ships operating off the California coast. The results of these observations are published in the "Summary of Synoptic Meteorological Observations (SSMO), North American Coastal Marine Areas - Revised, Pacific Coast, Volume 5 (1976)." Santa Cruz is within Area 35, shown in figure B7.

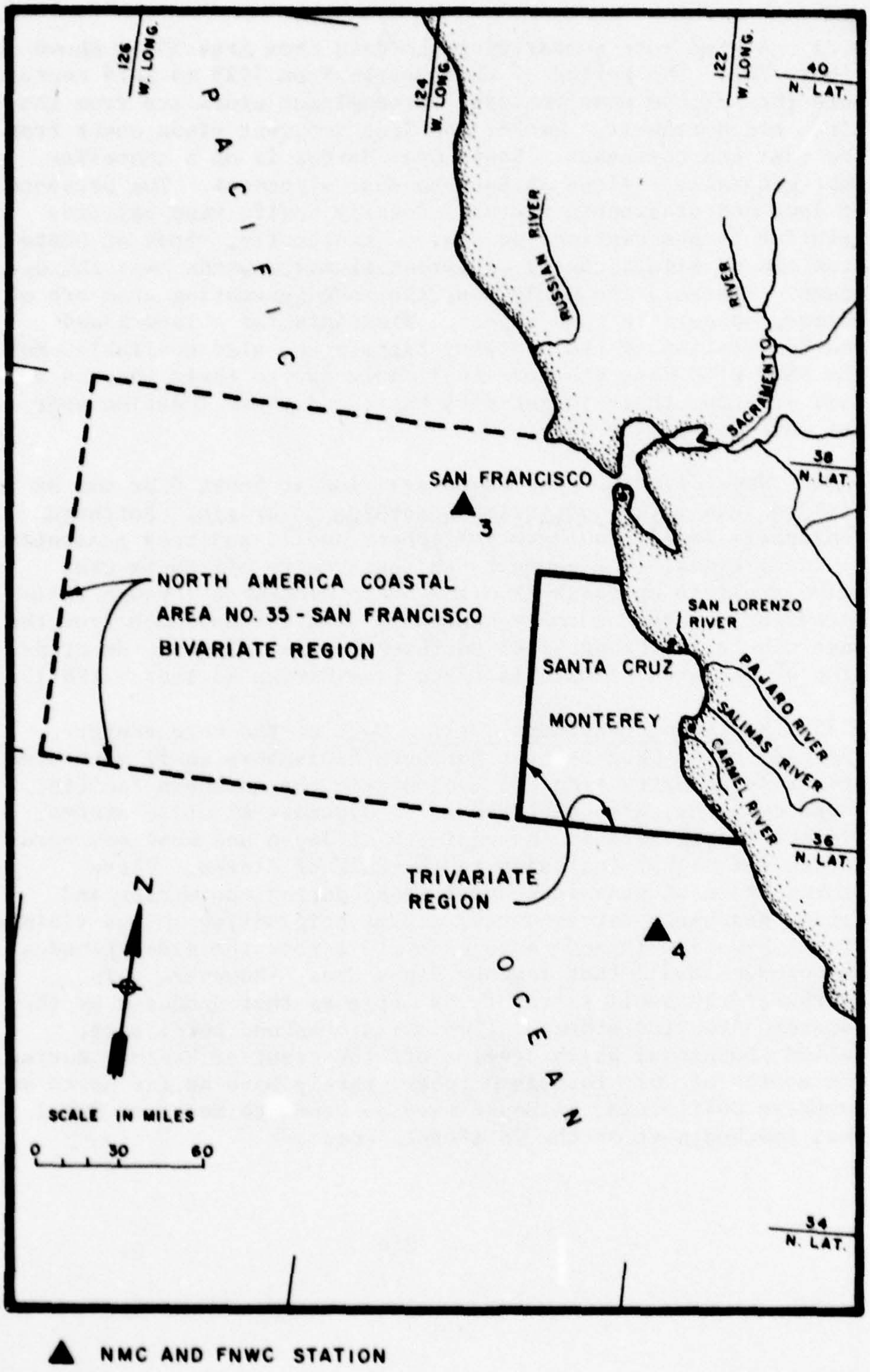
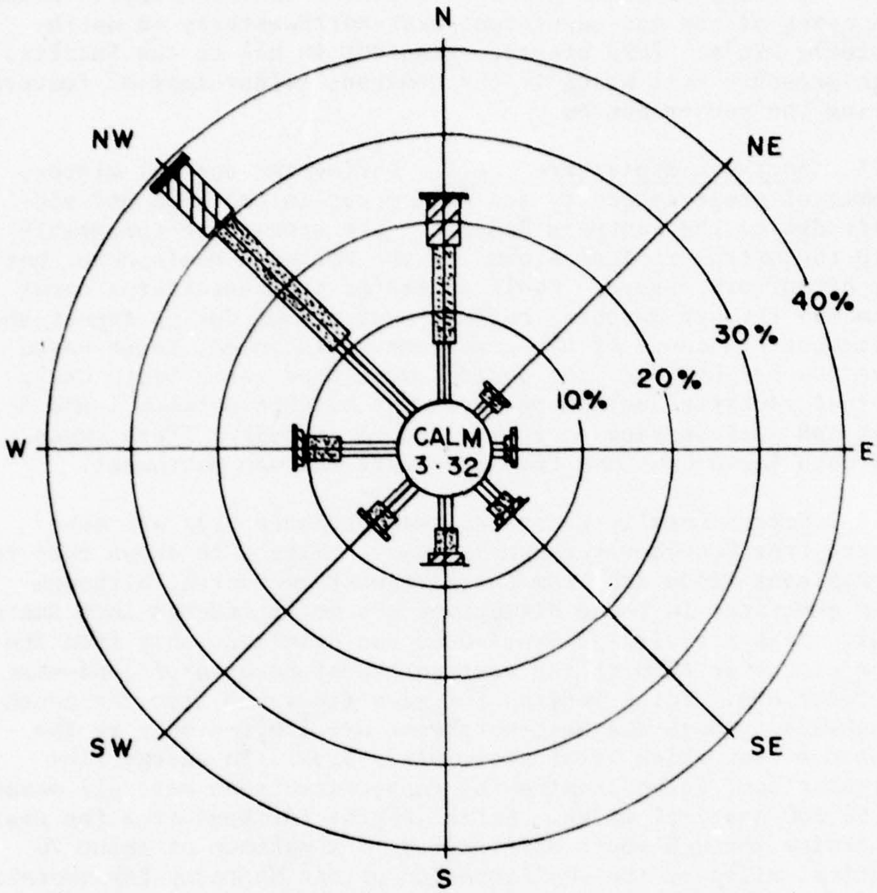


FIGURE B7. SSMO REGIONS

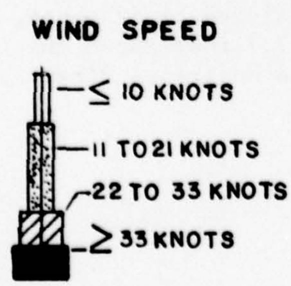
B.21 A wind rose summarizing the data from Area 35 is shown in figure B8. The period of observation from 1935 to 1974 contains more than 58,000 observations. Predominant winds are from the north and northwest. Weaker and less frequent winds occur from the east and northeast. Santa Cruz Harbor is on a shoreline that generally follows an east-to-west alignment. The presence of land and orographic features locally modify wind patterns relative to observations at sea. Consequently, winds at Santa Cruz can be significantly different from the winds over the open ocean. However, the winds over the wave-generating area are of primary concern in this report. Wind data for a land-based weather station at the Monterey Airport are also available, but the SSMO wind data are more applicable due to their absence of land effects, their longer data base, and their location over the wave-generating area.

B.22 Wave Climate. The waves arriving at Santa Cruz can be divided into three categories according to origin: Northern Hemisphere swell, Southern Hemisphere swell, and seas generated by local winds. The geometry and bathymetry off Santa Cruz allow swell to approach from the south clockwise through west-northwest, whereas locally generated seas can approach from the east clockwise through west-northwest. The following description of the wave climate is taken from Marine Advisers (1961).

B.23 Northern Hemisphere Swell. Most of the wave energy reaching Santa Cruz is from Northern Hemisphere swell generated primarily by extra-tropical cyclones in the northern Pacific. These cyclones, also referred to as Japanese-Aleutian storms, typically originate in the vicinity of Japan and move eastward across the higher latitudes to the Gulf of Alaska. These storms are most prevalent and intense during the winter and spring seasons. Infrequently, storms originating in the vicinity of the Hawaiian Islands move eastward across the mid-latitudes and produce swell that reaches Santa Cruz. However, this southwesterly swell is rarely as large as that produced by the Japanese-Aleutian storms. Tropical storms and hurricanes, called chubascos, which develop off the coast of Mexico, during the months of July through October, rarely move as far north as Southern California, although they do generate moderate swell that reaches most of the California coast.



WIND ROSE FOR SAN FRANCISCO
AREA 35



PRIMARY PERIOD OF RECORD
1935 — 1974
OVERALL PERIOD OF RECORD
1854 — 1974
58,544 OBSERVATIONS
SOURCE
SSMO (1976)
SAN FRANCISCO
AREA 35

FIGURE B8.

B.24 A steep pressure gradient over the eastern Pacific basin can cause strong and persistent west-northwesterly to northwesterly winds. This pressure gradient is due to the Pacific high-pressure cell which is the dominant meteorological feature during the summer months.

B.25 Southern Hemisphere Swell. During the austral winter, storms of great intensity and size occur in the high and mid-latitudes of the southern Pacific. The storms are comparable with the extra-tropical storms of the Northern Hemisphere, but are often more severe. Swell generated by these storms occur from May through October, but are most common during August and September. Because of the great decay distances, these waves have low heights and long periods when they reach Santa Cruz. Typical southern hemisphere swell has heights between 1 and 3 feet and periods ranging from 13 to 21 seconds. These waves approach Santa Cruz are from the south through southwest.

B.26 Seas. Locally generated seas at Santa Cruz are most severe from December through February. Figure B8 shows that the predominant winds are from the northwest and north, although seas generated in these directions are not refracted into Santa Cruz. Seas arriving at Santa Cruz can originate only from the east clockwise through the west-northwest because of land-mass restrictions. Fetch lengths for seas generated from the south clockwise through the west-northwest are limited only by the distance over which local storm winds blow. In energy-flux calculations, fetch lengths for these directions were all assumed to be 100 nautical miles. Fetch lengths for seas from the east clockwise through south are limited to a maximum of about 20 nautical miles by the configuration of the Monterey Bay shoreline. Refraction of waves over the irregular bathymetry of the Santa Cruz area has the effect of reducing seas approaching from the east or the west relative to waves approaching from the more southerly directions.

B.27 Tsunami. The first tsunami reported in the Santa Cruz area occurred on April 1, 1946. It consisted of 12 waves, the largest of which raised the tide level 7.7 feet above the predicted tidal stage. The water at the Municipal Pier

surged for three days following the arrival of that tsunami. In 1960 a six-foot rise in water level due to a tsunami generated by the Chilean earthquake was recorded, although no significant damage occurred at Santa Cruz. The 1964 Alaskan tsunami raised the water surface ten feet above the predicted tide level in Santa Cruz Harbor. As a result of this tsunami, a hydraulic dredge used for maintenance was torn from its moorings, and it sank after striking the east jetty. Thus, despite its apparently sheltered position, Santa Cruz is susceptible to damage by tsunamis approaching from the northwest. In fact, the runup at Santa Cruz in 1964 was twice that experienced at Monterey Harbor. This phenomenon is possibly due to refraction induced by the deep submarine canyon which bisects the bay.

B.28 Seiches and Currents. Two types of currents that occur in the Santa Cruz area are caused by long-period waves, or seiches, and unidirectional currents in the offshore surface. Seiches with periods ranging from 40 seconds to greater than four minutes have been measured in Monterey Bay by wave gages at Monterey Harbor and Moss Landing. These long-period waves have been reported to occur at the Santa Cruz area, but have not been measured by instruments. The estimated wave height of these seiches ranges from 0.5 feet to one foot.

B.29 Offshore surface currents in Santa Cruz Cove measured in 1925 and 1926 at the 40-foot depth contour were generally from west to east. Currents measured at the seaward end of the municipal pier were always directed shoreward. The maximum current velocity recorded during the study was 1.2 feet per second. No other current studies are available for the immediate vicinity of Santa Cruz Harbor.

LITTORAL PROCESSES

B.30 Wave Transport. Littoral transport is the movement of littoral drift due to the interaction of waves, winds, and currents with sediments. Modes of sediment motion are suspension, saltation, and bed load. Wave action moves bottom materials up, down, towards, and away from the coast, tending to establish

a beach and offshore profile that is in equilibrium with forces due to water motion and gravity. During this process, a winnowing action occurs which carries the finer materials farther offshore and deposits the coarser particles on the beach. Large, steep waves tend to erode the beach and move materials offshore, whereas moderate- and low-height waves tend to move material back towards the beach. The direction in which the sediments move parallel to the beach is largely determined by the direction of the incoming waves. This direction can vary from day to day and cause littoral materials to move both up and down the coast. Longshore transport occurs when beach material is moved along a shoreline by the skewed uprush and backwash of waves arriving obliquely to the shoreline; just offshore, longshore transport results from longshore currents. Movement occurs primarily between the breaker line and the wave runup limit. Material is also transported when waves stir the bottom materials into suspension and bottom currents transport the material. The absolute sum of all longshore littoral movements over a year gives a gross annual drift, whereas a vector sum of the longshore movements results in a net annual drift. The direction of the net longshore transport is termed "downdrift", and the opposite direction, "updrift". At Santa Cruz, the downdrift direction is to the east.

B.31 The longshore movement of materials is interrupted when a jetty or channel is constructed across the zone of littoral transport. A long jetty will impound the net littoral transport on the updrift side of the structure, and an equal amount of material will theoretically erode from the downdrift beach. A channel will tend to trap the gross littoral transport due to the decrease in water turbulence a channel provides.

B.32 Wind Transport. Aeolian transport refers to the process whereby beach materials are transported by wind. This process is often evidenced by the presence of sand dunes on the backshore. Migration of these dunes landward can result in a loss of sand from the active littoral zone. Prior to the construction of the Harbor, there was no evidence of aeolian transport of significant amounts of beach material, although small dunes were noted on the downcoast side of some streams and beaches in the study area. After the construction of the jetties, windblown sand accumulated against the backshore part of the west

jetty and material overtopped the jetty, subsequently settling in the channel.

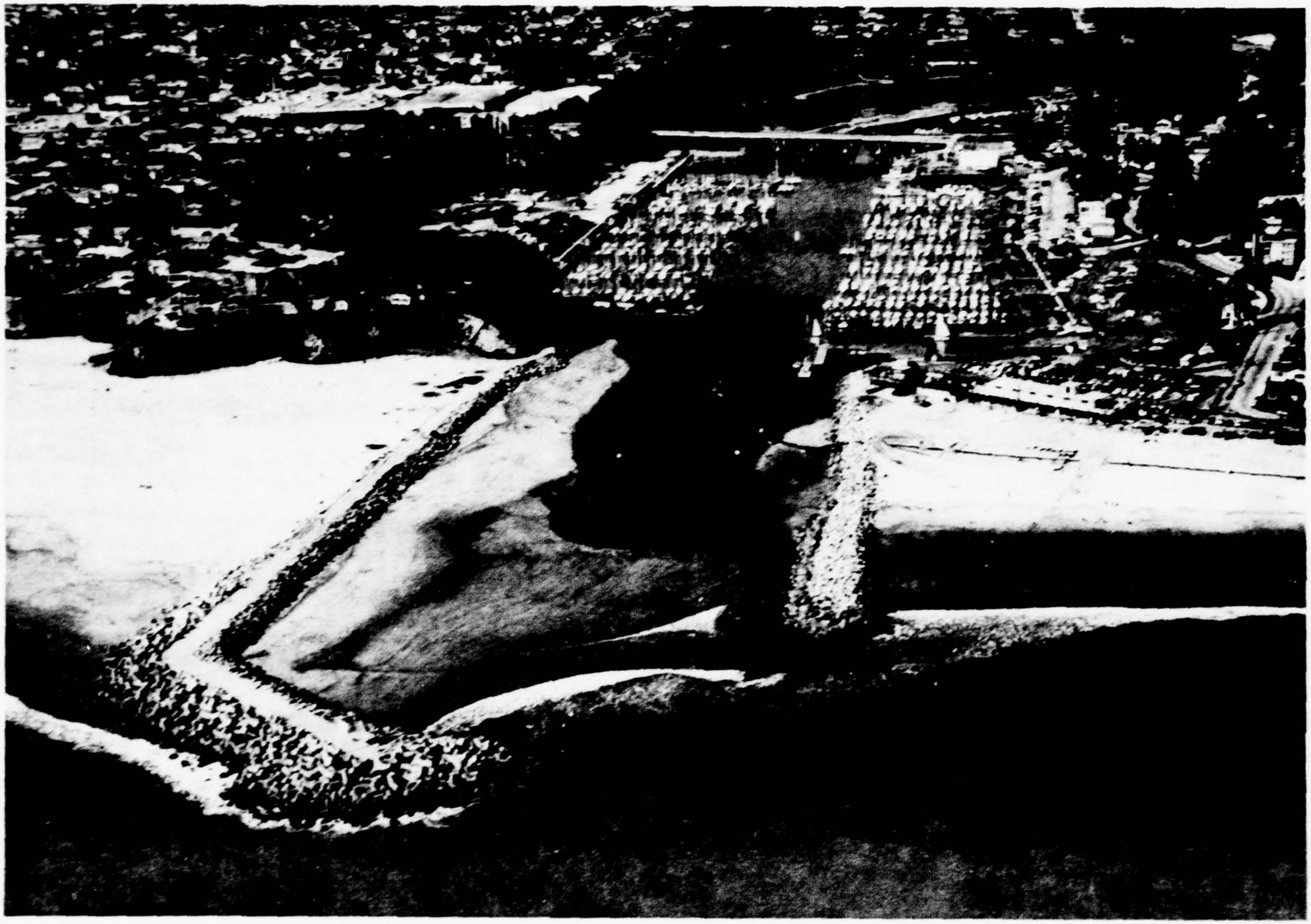
B.33 Inlet Effects. One of the objectives of constructing jetties is to control the shoaling of a channel by littoral transport. Given an adequate tidal prism in the harbor, the channel will be maintained by the scouring action of the tidal flows. The Santa Cruz Harbor jetties, as evidenced by the yearly shoaling of the channel, are not adequately preventing the infilling of the channel. In the first few years after construction of the jetties, significant shoaling of the inlet did not occur. Shoaling of the channel increased rapidly when the impounded fillet west of the west jetty aggraded to within 300 feet of the head of the jetty, suggesting that sand was bypassing around the end of the short west jetty into the entrance channel. This material remained in the inlet because the tidal flow velocities were not of sufficient magnitude to scour the excess sand. The tidal prism is not large enough to maintain the desired channel dimension.

STATEMENT OF THE PROBLEM

B.34 Construction of Santa Cruz Harbor and maintenance dredging have interfered with littoral transport in the vicinity of the Harbor and have altered both the updrift and downdrift beach configurations. The updrift beach between the San Lorenzo River and the west jetty has experienced significant accretion since the construction of the jetties. This accretion is due to the impoundment of sand against the west jetty, which has acted as a groin and tended to stabilize the updrift shoreline reach. The downdrift beach, between the east jetty and Black Point, has also experienced long-term accretion since the construction of the jetties. This accretion may be due in part to the construction of the jetties between two natural headlands, the rock projection east of the San Lorenzo River and Black Point, which define this shoreline reach. The new headland created by the jetties reduced the distance between headlands by forming two cells, each with increased capability to retain sand. This has more effectively stabilized the shoreline. The timing of maintenance sand-bypassing dredging

after the major winter storm may also contribute toward the accretion on the downdrift beach. One year after the completion of the Harbor, shoaling occurred in the entrance channel and dredging of the harbor was necessary. In subsequent years, the entrance channel has shoaled during the winter season; dredging in the spring has been required to maintain a navigation channel. Photograph B1 shows the channel at a low tide at the beginning of a dredging episode. This procedure has resulted in closure of the harbor between December and April until 1978 when the dredging practice was changed to maintain the channel throughout the year.

B.35 The problem is to identify the coastal processes occurring in the study area and to determine which processes are directly responsible for the shoaling of the harbor entrance. The possible causes of the shoaling problem are; longshore sediment transport around the jetties, leakage of sand through the jetties, aeolian transport of sediment into the channel, onshore transport of offshore deposits, and tidal transport. The purpose of defining the processes is to provide criteria for developing a method of mitigating the shoaling and maintaining a safe, year-round entrance channel.



Photograph B1: Shoaled Entrance Channel

SECTION C

LITTORAL PROCESSES

CONTENTS

CONTENTS
(Continued)

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(Continued)

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SECTION C

LITTORAL PROCESSES

C.1 The purpose of this section is to present and analyze the available data on littoral processes in order to define the mechanism of shoaling in Santa Cruz Harbor. The wave climate is described to form a basis for wave energy-flux calculations and for consideration in formulating and evaluating maintenance or bypassing procedures. Wave energy-flux calculations are made to estimate the potential for littoral transport. Bathymetric surveys and dredging records are analyzed to determine historic shoaling rates and to give insight into the shoaling mechanism. Thus, based on available data and theory, the shoaling process is synthesized.

WAVE CLIMATE

C.2 General. Several wave data sets are available for Santa Cruz. Characteristics of waves had been measured in shallow water visually and by wave gages; hindcasts had been made for wave characteristics at deep-water stations in offshore areas. Four sources of measured shallow-water waves included visual Littoral Environmental Observations (LEO), a Marine Advisers wave gage off the Santa Cruz west jetty, a wage gage off the municipal pier, and a wave-gage array installed by the Corps of Engineers and State of California, Department of Navigation and Ocean Development, as part of the California Coastal Engineering Data Network. Four sources of deep-water wave statistics included shipboard observations from the Summary of Synoptic Meteorological Observations (SSMO), and three hindcast studies listed below:

1. "Wave Statistics for Seven Deep-Water Stations Along the California Coast", National Marine Consultants, December 1960;

2. "Deep-Water Wave Statistics for the California Coast," Meteorology International Incorporated, February 1977; and

3. "A Statistical Survey of Ocean Wave Characteristics in Southern California Waters", Marine Advisers, January 1961.

C.3 LEO Data. Visual observations of littoral processes were taken by rangers at Twin Lakes State Beach on the east side of Santa Cruz entrance channel. Daily visual observations of wave height, period, breaker type, and direction of approach were taken during a period from September 1, 1968, to February 16, 1972. Wave heights observed ranged up to 11 feet and wave periods ranged up to about 40 seconds. Wave directions were classified into 45-degree sectors with the majority of the waves approaching from the southwest. The total number of wave observations was 786. The largest and smallest number of observations were taken in September and August, respectively.

C.4 Marine Advisers Wave Gage. Marine Advisers installed and monitored a wave-pressure gage located 425 feet southeast of the head of the west jetty in a depth of 20 feet MLLW. The wave gage operated between October 1, 1963, and June 10, 1968. The data were reduced manually. Within this time period, the maximum measured significant wave height was 13 feet in December 1967.

C.5 Municipal Pier Wave Gage. A step-resistance type gage was mounted in February 1968 on the seaward end of the Santa Cruz Municipal Pier, about 1 mile west of the project site, and operated until May 1972. This gage was installed with the objective of measuring extreme wave events; consequently, the data were not reduced on a routine basis.

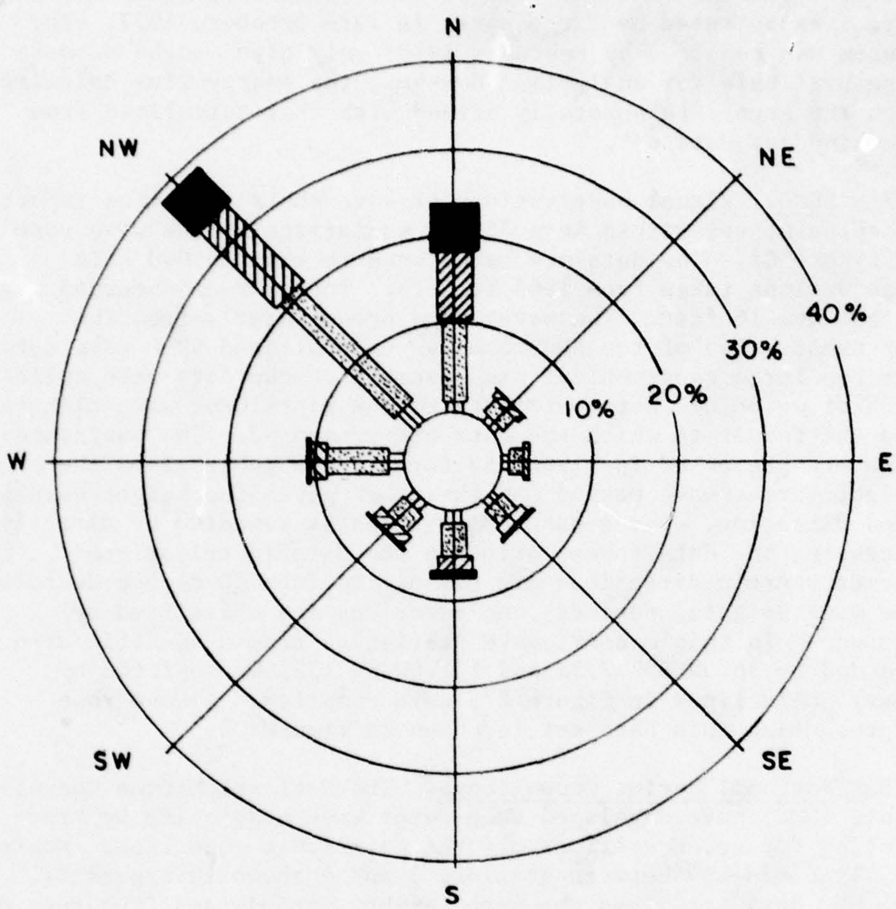
C.6 California Coastal Engineering Data Network. A wave gage station was installed at Santa Cruz Harbor in September 1977 as part of the California Coastal Engineering Data Network sponsored jointly by the Department of Navigation and Ocean Development and the U. S. Army Corps of Engineers. The system comprised four pressure transducers mounted at the corners of a 6-meter-square frame connected by submerged cable to a shore-based recorder. The system measures the momentum flux and slope of the wave-induced pressure field. The wave height, wave-energy spectra, and longshore transport rate were calculated by linear wave theory using the data from the slope-array system and some empirical data.

Initially installed in 20 feet of water, the transducer elements were incapacitated by large waves in late October, 1977. The system was repaired by February 1978; only five months of data were available for analysis. However, the energy flux calculated from the gage data generally agreed with that calculated from the hindcast data set.

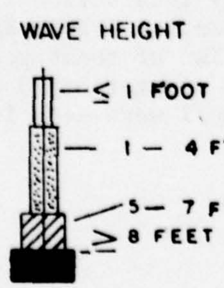
C.7 SSMO. Visual observations of wave characteristics reported by ships at sea within Area 35 are summarized by the wave rose in figure C1. The data are based on more than 16,000 ship observations taken from 1963 to 1974. The maximum observed wave height was 16 feet. The waves were predominantly from the northwest. Two of the shortcomings of published SSMO wave data are the large geographical area over which the data were collected, much of which did not contribute to the Santa Cruz wave climate, and the format in which the data are presented. The published data are presented in bivariate format; one table gives the height versus wave period and the other gives the height versus wind direction. These shortcomings can be remedied by directly accessing the data tapes, printing the data in trivariate format wherein directions are classified into 30-degree sectors and wave heights, periods, and durations are classified by seasons. In this manner, wave statistics from a specific area bounded by 36.0N by 37.3N and 121.0W by 122.9W, depicted by heavy solid lines in figure B7, were compiled. A wave rose representing this data set is shown in figure C2.

C.8 National Marine Consultants. The National Marine Consultants (NMC) have developed deep-water wave statistics by hindcasting for seven stations off the California coastline. Monterey Bay lies mid-way between stations 3 and 4 shown in figure B7. The NMC data set gives the wave height, period, and frequency of occurrence of deep-water, Northern Hemisphere-generated swell and of locally generated seas at each station by 22.5-degree sectors.

C.9 U.S. Navy Fleet Numerical Weather Central. Deep-water wave statistics were developed by meteorology International, Inc. from the U.S. Navy Fleet Numerical Weather Central (FNWC) data for six stations off the California coast. Two of these stations share the same locations and station numbers as stations 3 and 4 of the NMC report. Hindcasts of seas and swell were made from



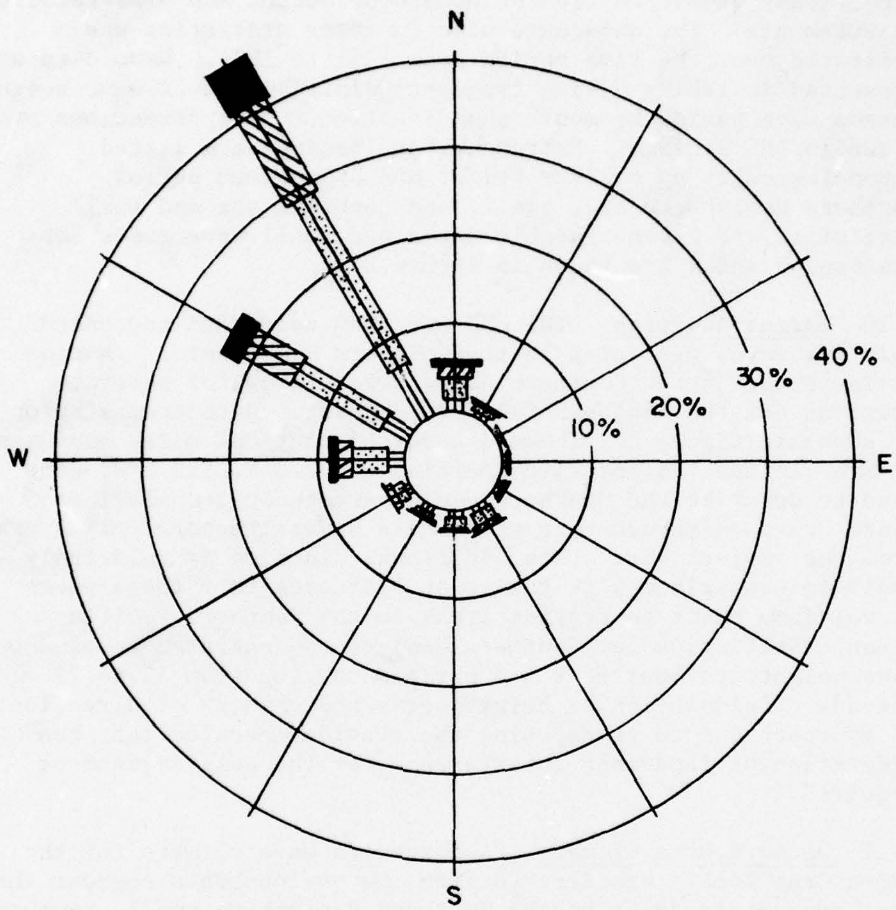
WAVE ROSE FOR SAN FRANCISCO



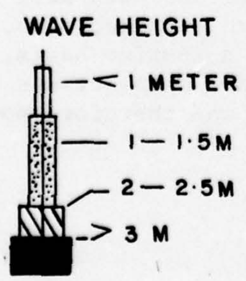
OVERALL PERIOD OF RECORD
1963 — 1974

16,485 OBSERVATIONS
SOURCE
SSMO (MAY 1976)
AREA 35 — SAN FRANCISCO

FIGURE C1



WAVE ROSE FOR SANTA CRUZ



OVERALL PERIOD OF RECORD
1949 — 1973

SOURCE:
SSMO TRIVARIATE DATA

OBSERVATION GRID
36.0° THRU 37.3° NORTH LATITUDE
121.0° THRU 122.9° WEST LONGITUDE

FIGURE C2

wind fields developed from shipboard-barometer and wind-velocity measurements. The data base used in these statistics was collected over the time period from 1951 to 1974. Wave data are presented in tables giving frequency distribution of wave height versus wave period by month and direction. Wave directions are given in 10° sectors. Extreme-event listings are listed chronologically by maximum height and by maximum period. Northern Hemisphere sea, swell, and combined sea and swell statistics are given. Combined sea and swell wave roses for Stations 3 and 4 are shown in figure C3.

C.10 Marine Advisers. The NMC and FNWC wave data accounted only for waves generated in the Northern Hemisphere. Marine Advisers considered Southern Hemisphere storms for several stations off the Southern California coast. Data from station A, shown in figure C4, located about 65 nautical miles southwest of San Clemente Island with coordinates 32.3°N, 119.6°W, were used to describe the Southern Hemisphere-generated swell at Santa Cruz. Although this station is several hundred miles away from the project site, this additional distance is relatively small in comparison with the great distances that these waves travel from their generating areas in the southern Pacific Ocean. Statistics for Southern Hemisphere-generated waves show wave heights to four feet and periods ranging from 12 to 22 seconds. Calculation of height decay and changes of direction of approach due to transposing the station revealed that consideration of land-mass interference was the only adjustment required.

C.11 Derived Wave Climate. A composite wave climate for the Santa Cruz locale was derived from the various data sources that most accurately describe the Northern Hemisphere swell, Southern Hemisphere swell and local sea. Figures C5 and C6 present wave-period histograms derived from the various sources. The SSMO wave data sets were deleted after comparison of wave periods which indicated that the SSMO wave periods tended to reflect short-period, steep wind waves and to ignore the swell which refracts into the Santa Cruz embayment. The LEO data were too sparse to allow development of a viable wave climate; also, the wave-gage measurements were not reduced on a routine basis, nor was direction of approach reported. The FNWC data set was not published at the time of this analysis and was therefore not

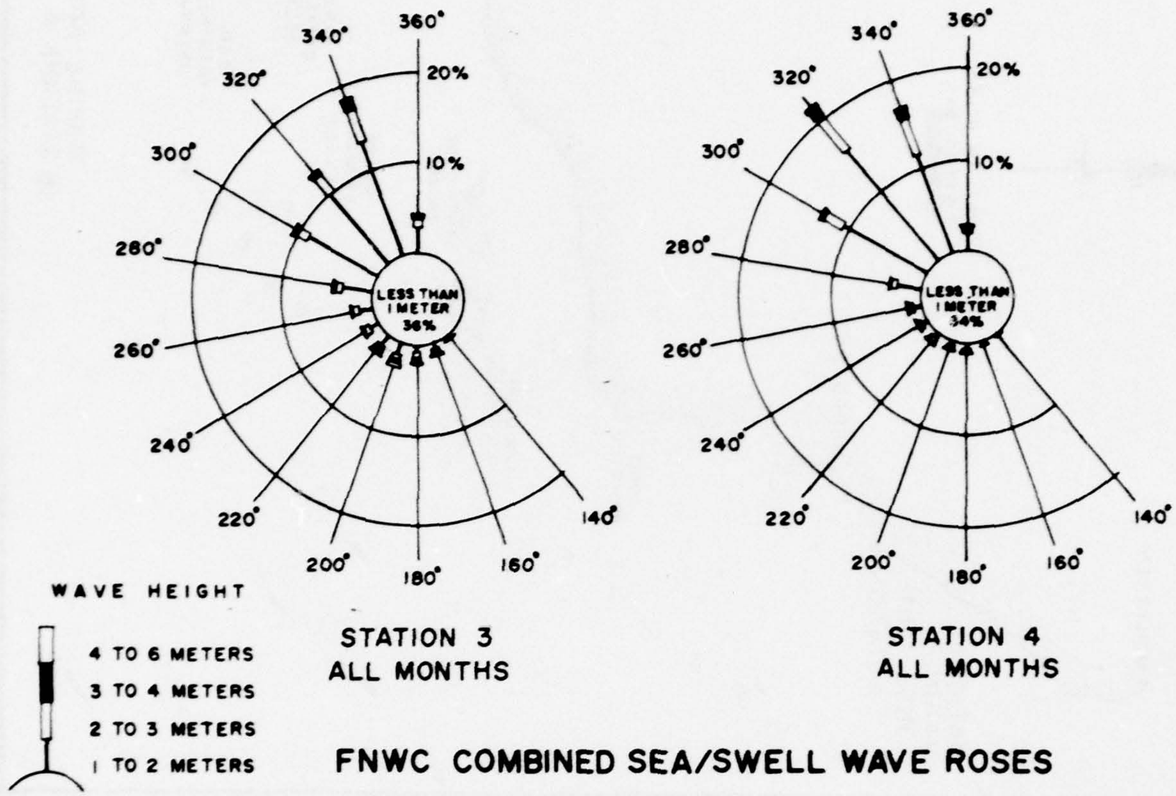

360°
340°
320°
300°
280°
260°
240°
220°
200°
180°
160°
140°
20%
10%
LESS THAN 1 METER 36%
LESS THAN 1 METER 34%
WAVE HEIGHT
4 TO 6 METERS
3 TO 4 METERS
2 TO 3 METERS
1 TO 2 METERS
STATION 3
ALL MONTHS
STATION 4
ALL MONTHS
FNWC COMBINED SEA/SWELL WAVE ROSES

Figure C3.

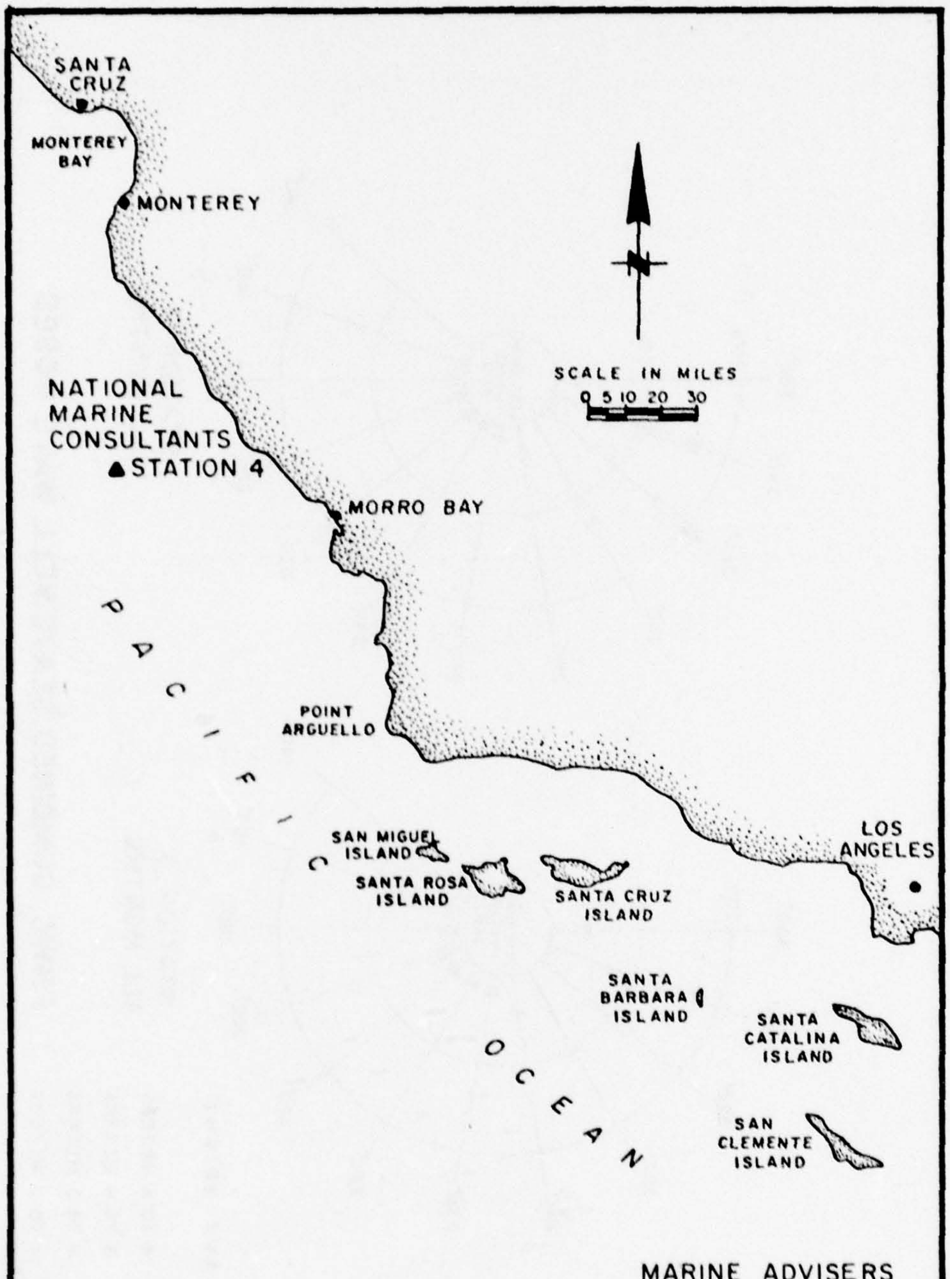


FIGURE C4. STATION A

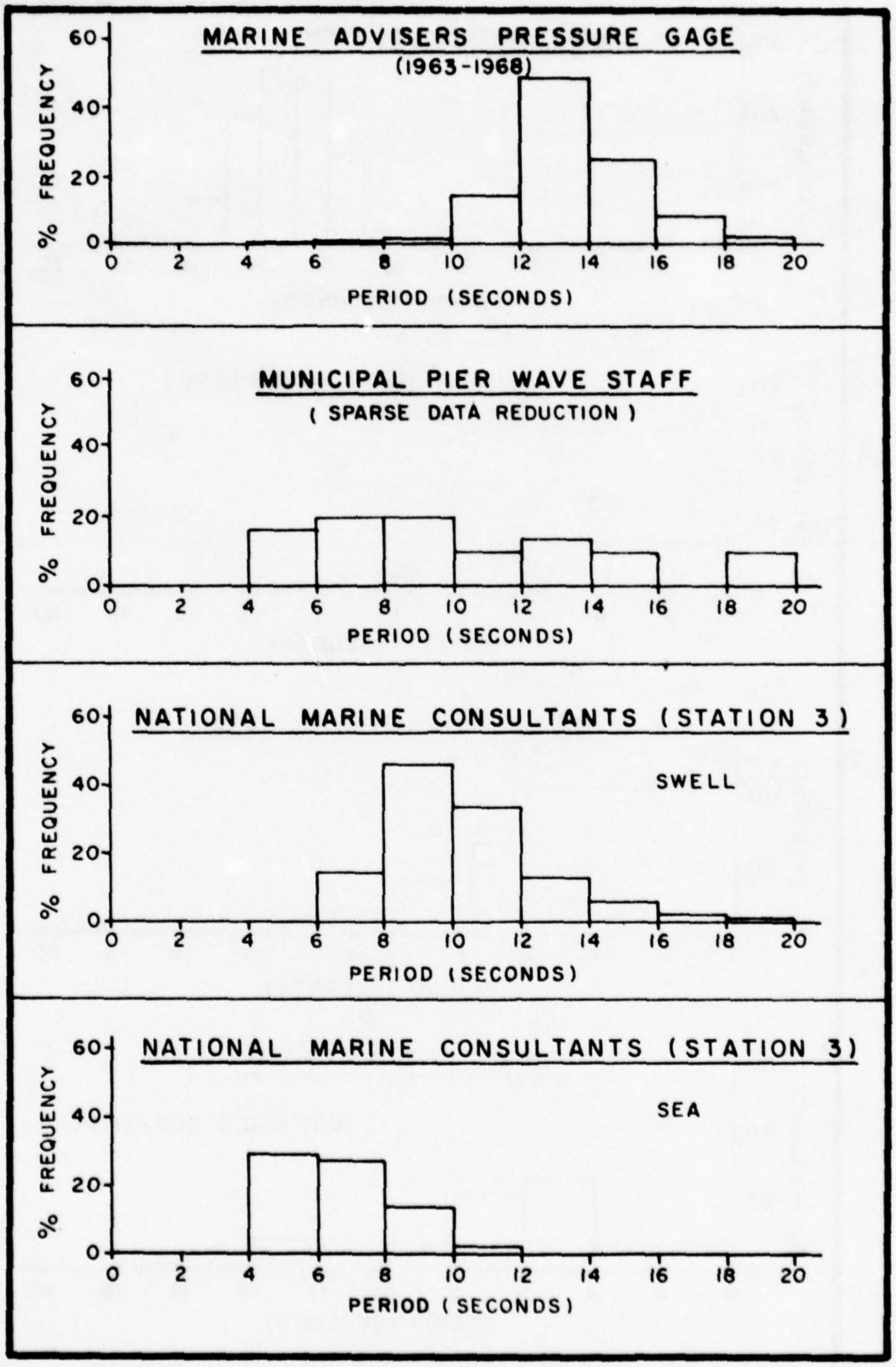


FIGURE C5.

WAVE-PERIOD HISTOGRAMS

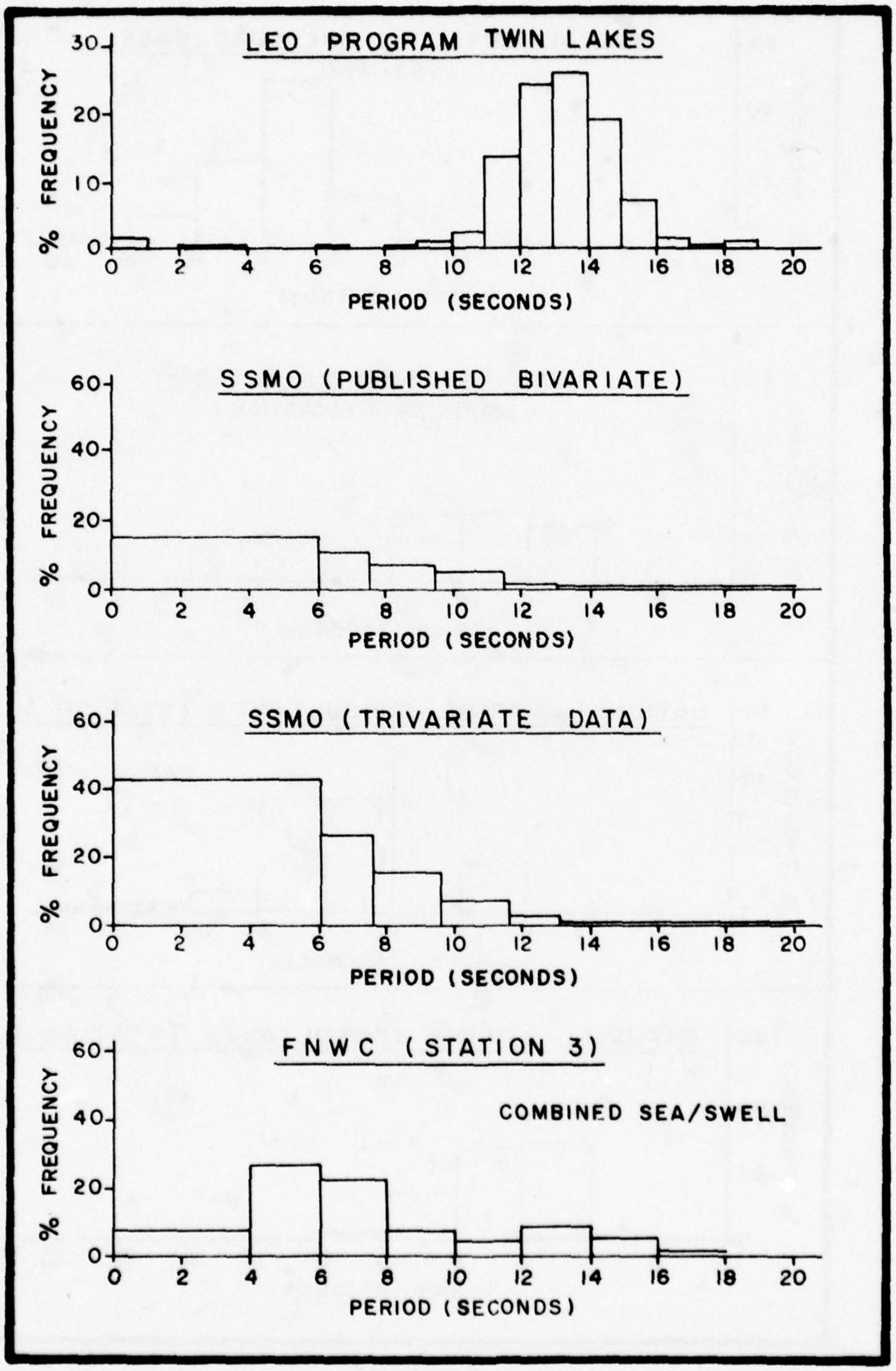


FIGURE C6.

WAVE-PERIOD HISTOGRAMS

considered further for the energy-flux calculations. The slope-array gage data were of too short a duration to be used for an average annual prediction of the wave climate. Therefore, the wave climate was compiled using the hindcast National Marine Consultants description of Northern Hemisphere swell and the Marine Advisers description of the Southern Hemisphere swell. Local seas were described using the National Marine Consultants data set for waves from the south-southeast to west and were hindcast using the SSMO wind rose for effective fetches exposed to the east and southeast. Although this wave climate description is based on hindcast and sparse measurements of southern swell, it is the most reliable and accurate description available at this time. The wave gage system recently installed in Monterey Bay should provide more accurate data after a few years of data collection and analysis.

C.12 The wave climate for the harbor site was derived by refraction and shoaling analysis of the deep-water waves. Tables C1 and C2 show the refraction coefficients and directions of wave approach at the 18-foot depth contour in the vicinity of the Harbor. Swell more northerly than northwest had refraction coefficients less than 0.2 and were deleted. Seas from the northwest and north of northwest were also deleted for similar reasons. Swell from directions east of south were deleted due to shadowing by Pt. Pinos on the south flank of Monterey Bay. Monthly cumulative wave-height distributions, shown in figures C7 through C18, compare the measured waves at the wave gages with the transformed waves. The graphs present cumulative distributions of sea, swell and the joint occurrence of sea and swell. February has the greatest occurrence of high waves. The wave-height distributions from the municipal pier exhibited a wide scatter in two years; this scatter bracketed the derived wave climate. The Marine Advisers gage, which reported a single height and period as opposed to the combined sea and swell, more closely duplicates the individual distributions for long-recurrence-interval waves. The cumulative probability distribution of combined sea and swell was developed assuming that seas and swells are independent events and that some simultaneous arrivals of sea and swell superpose linearly to produce a locally greater wave height.

TABLE C1

REFRACTION COEFFICIENTS

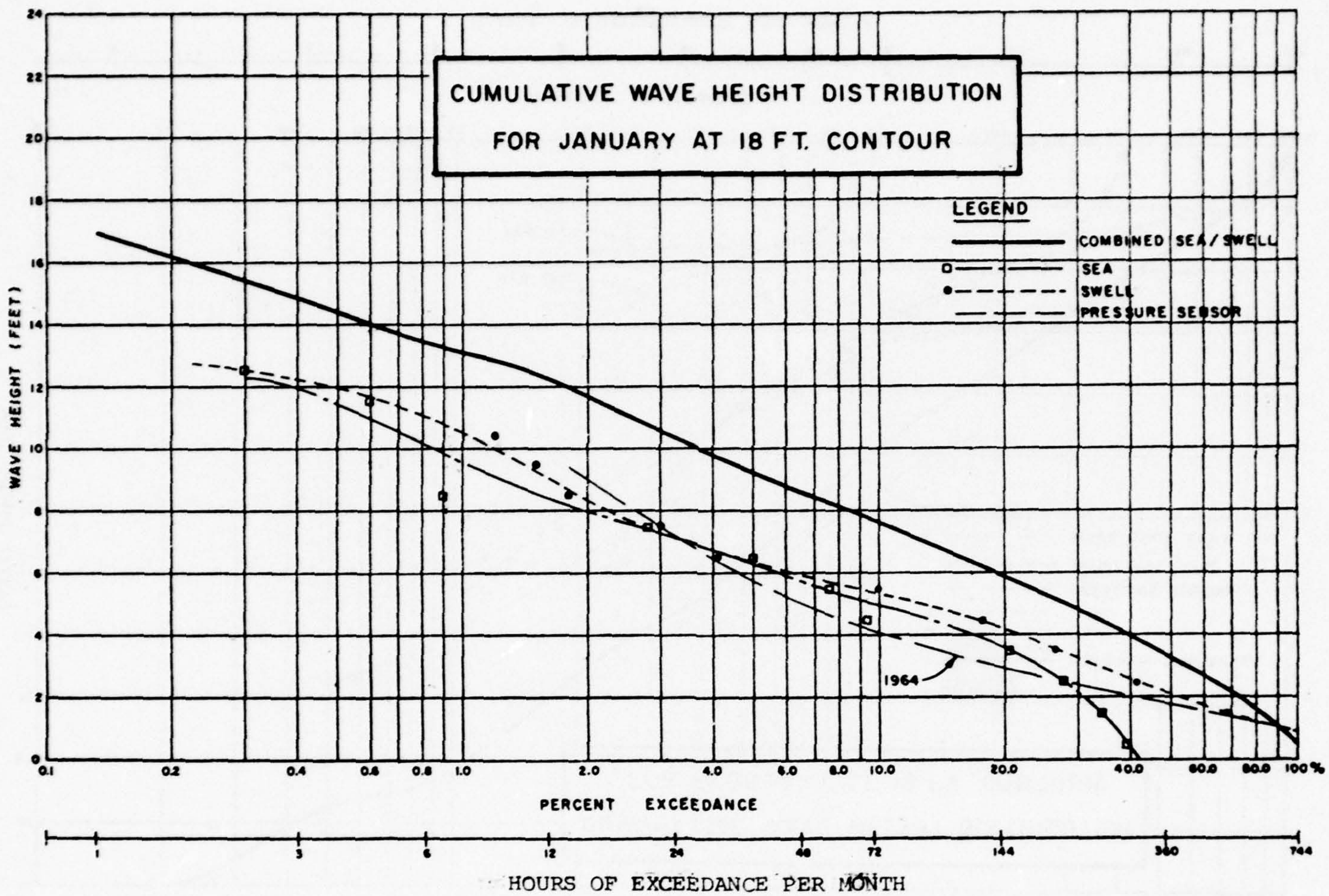
AZ* T	135	157.5	180	202.5	225	247.5	270	292.5
5	.90	.94	1.06	1.00	.94	.61	.15	.00
7	.89	.93	.96	.96	.95	.61	.20	.00
9	-	.91	.93	.86	.96	.61	.30	.15
11	-	-	.90	.88	.89	.64	.39	.20
13	-	-	.91	.91	.81	.65	.48	.24
15	-	-	.92	.91	.84	.88	.43	.26
17	-	-	1.10	.90	.87	.92	.39	.29
19	-	-	.89	.81	.89	.95	.62	.24
21	-	-	1.22	-	-	-	-	-

TABLE C2

WAVE RAY AZIMUTHS AT 18' CONTOUR

AZ* T	135	157.5	180	202.5	225	247.5	270	292.5
5	149°	166°	183°	201°	214°	217°	230°	-
7	160°	171°	184°	197°	207°	211°	230°	-
9	-	175°	184°	195°	201°	205°	213°	214°
11	-	-	185°	195°	202°	201°	200°	214°
13	-	-	184°	196°	203°	205°	208°	214°
15	-	-	184°	195°	202°	204°	204°	212°
17	-	-	186°	193°	200°	203°	197°	209°
19	-	-	184°	189°	199°	202°	198°	215°
21	-	-	177°	-	-	-	-	-

*AZ is azimuth index relative to N.
T is wave period in seconds.



CUMULATIVE WAVE HEIGHT DISTRIBUTION
FOR JANUARY AT 18 FT. CONTOUR
LEGEND
COMBINED SEA/SWELL
SEA
SWELL
PRESSURE SENSOR
1964
WAVE HEIGHT (FEET)
0
2
4
6
8
10
12
14
16
18
20
22
24
0.1
0.2
0.4
0.6
0.8
1.0
2.0
4.0
6.0
8.0
10.0
20.0
40.0
60.0
80.0
100%
PERCENT EXCEEDANCE
1
3
6
12
24
48
72
144
360
744
HOURS OF EXCEEDANCE PER MONTH

FIGURE C7.

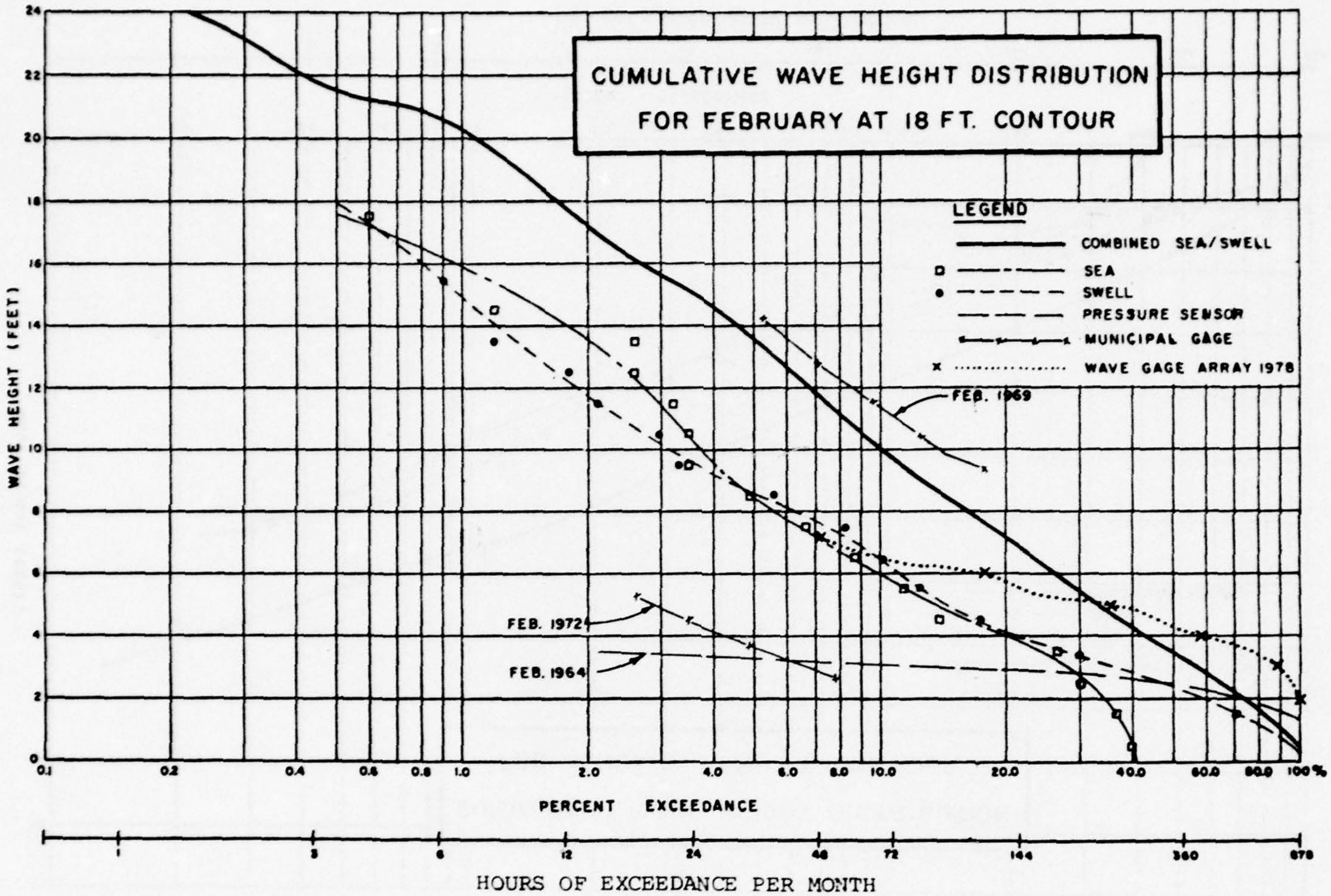
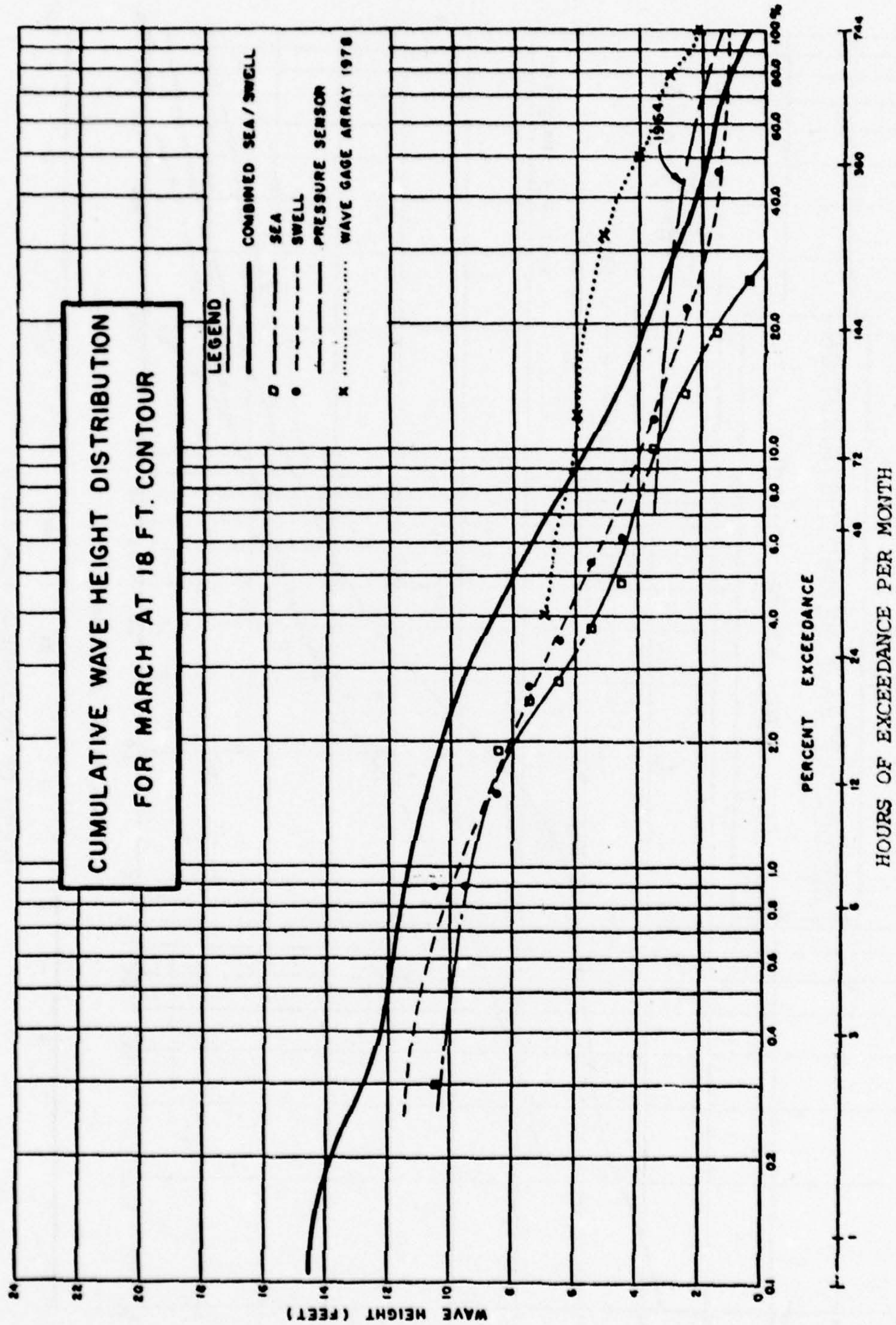
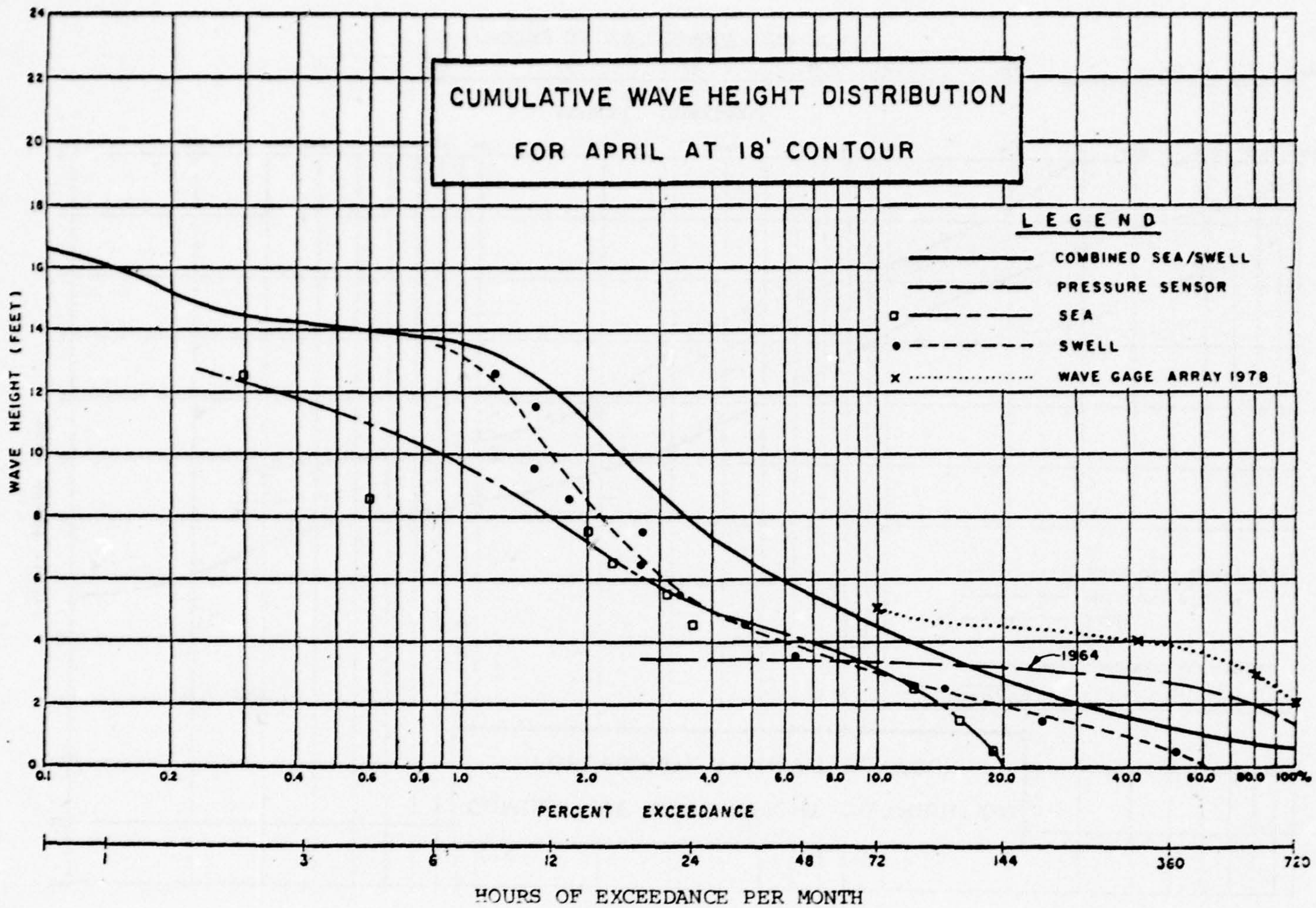

CUMULATIVE WAVE HEIGHT DISTRIBUTION
FOR FEBRUARY AT 18 FT. CONTOUR
LEGEND
COMBINED SEA/SWELL
SEA
SWELL
PRESSURE SENSOR
MUNICIPAL GAGE
WAVE GAGE ARRAY 1978
FEB. 1969
FEB. 1972
FEB. 1964
WAVE HEIGHT (FEET)
PERCENT EXCEEDANCE
HOURS OF EXCEEDANCE PER MONTH

FIGURE C8.



CUMULATIVE WAVE HEIGHT DISTRIBUTION
FOR MARCH AT 18 FT. CONTOUR
LEGEND
COMBINED SEA / SWELL
SEA
SWELL
PRESSURE SENSOR
WAVE GAGE ARRAY 1978
1964
WAVE HEIGHT (FEET)
PERCENT EXCEEDANCE
HOURS OF EXCEEDANCE PER MONTH
0.1
0.2
0.4
0.6
0.8
1.0
2.0
4.0
6.0
8.0
10.0
20.0
40.0
60.0
80.0
100%
1
3
6
12
24
48
72
144
360
744
0
2
4
6
8
10
12
14
16
18
20
22
24

FIGURE C9.



CUMULATIVE WAVE HEIGHT DISTRIBUTION
FOR APRIL AT 18' CONTOUR
LEGEND
COMBINED SEA/SWELL
PRESSURE SENSOR
SEA
SWELL
WAVE GAGE ARRAY 1978
1964
WAVE HEIGHT (FEET)
PERCENT EXCEEDANCE
HOURS OF EXCEEDANCE PER MONTH

FIGURE C10.

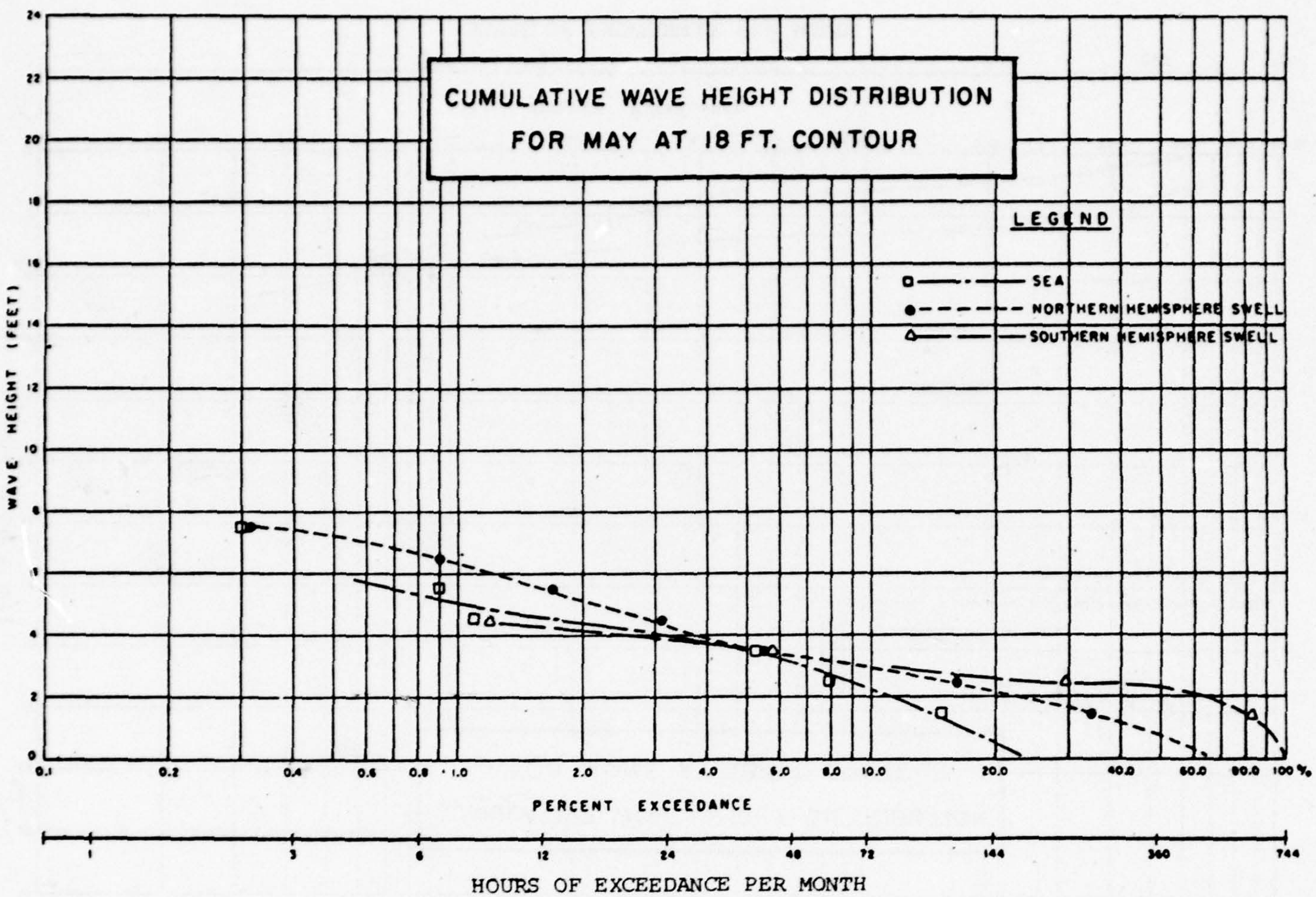

CUMULATIVE WAVE HEIGHT DISTRIBUTION
FOR MAY AT 18 FT. CONTOUR
LEGEND
SEA
NORTHERN HEMISPHERE SWELL
SOUTHERN HEMISPHERE SWELL
WAVE HEIGHT (FEET)
24
22
20
18
16
14
12
10
8
6
4
2
0
0.1
0.2
0.4
0.6
0.8
1.0
2.0
4.0
6.0
8.0
10.0
20.0
40.0
60.0
80.0
100 %
PERCENT EXCEEDANCE
1
3
6
12
24
48
72
144
360
744
HOURS OF EXCEEDANCE PER MONTH

FIGURE C11.

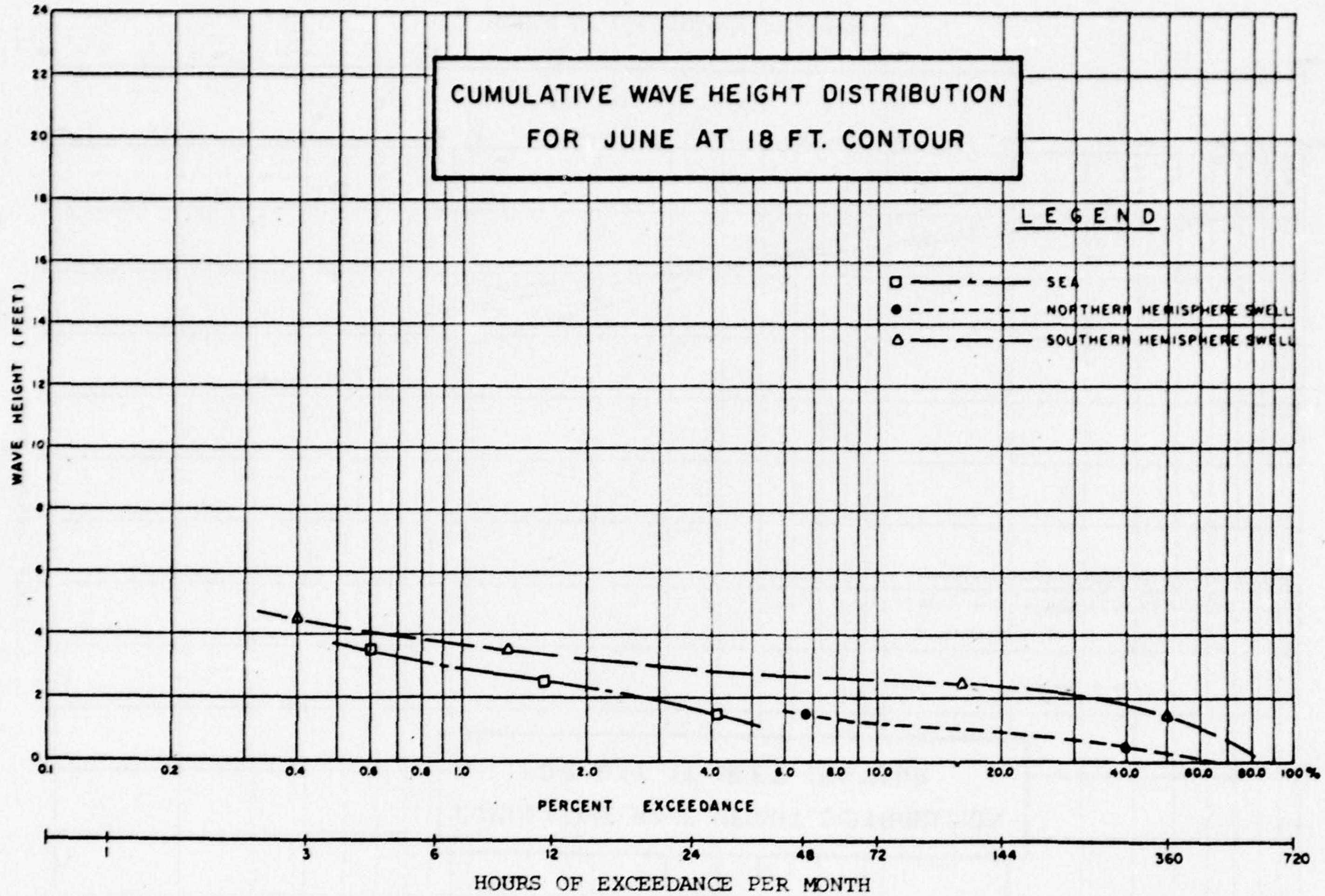
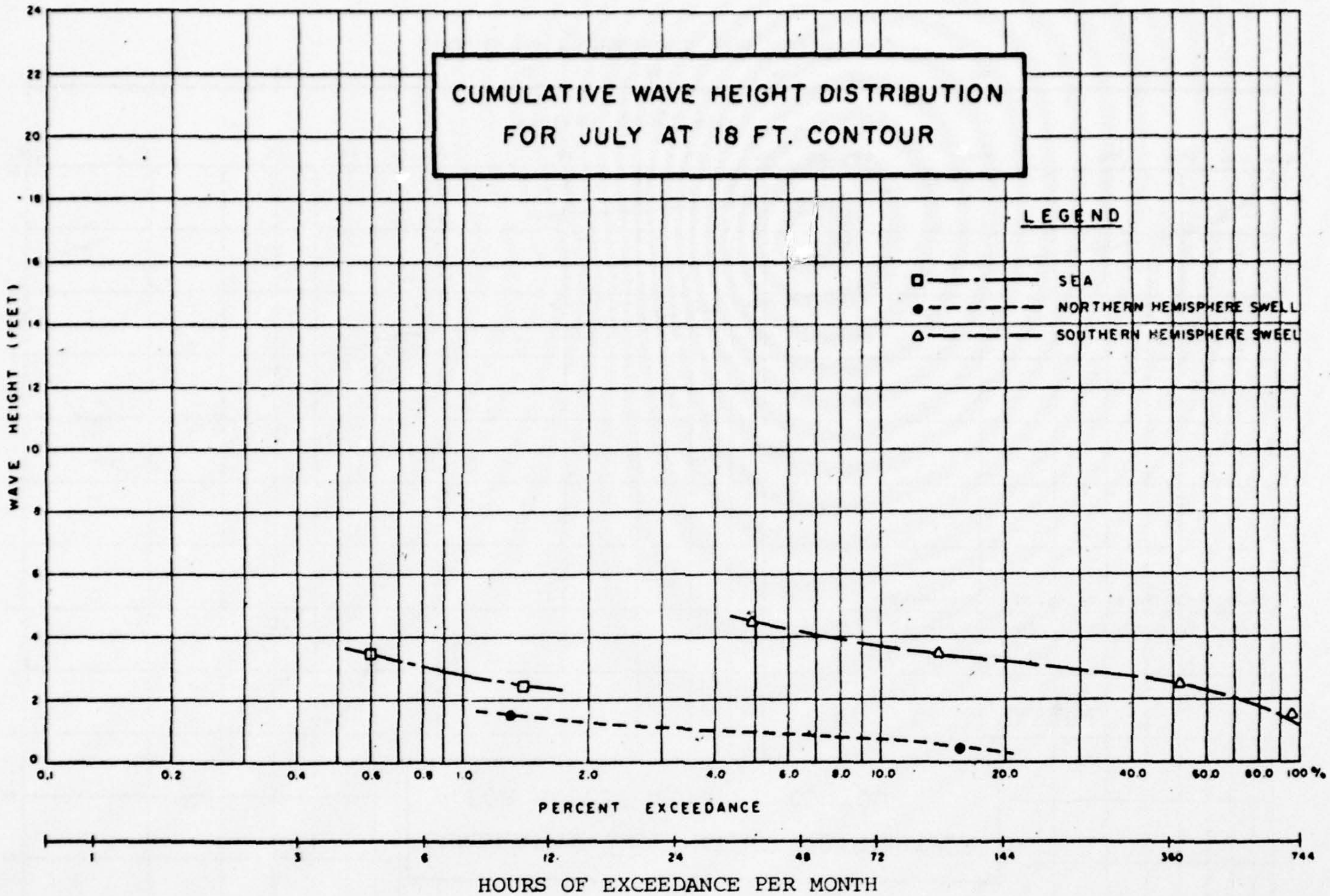
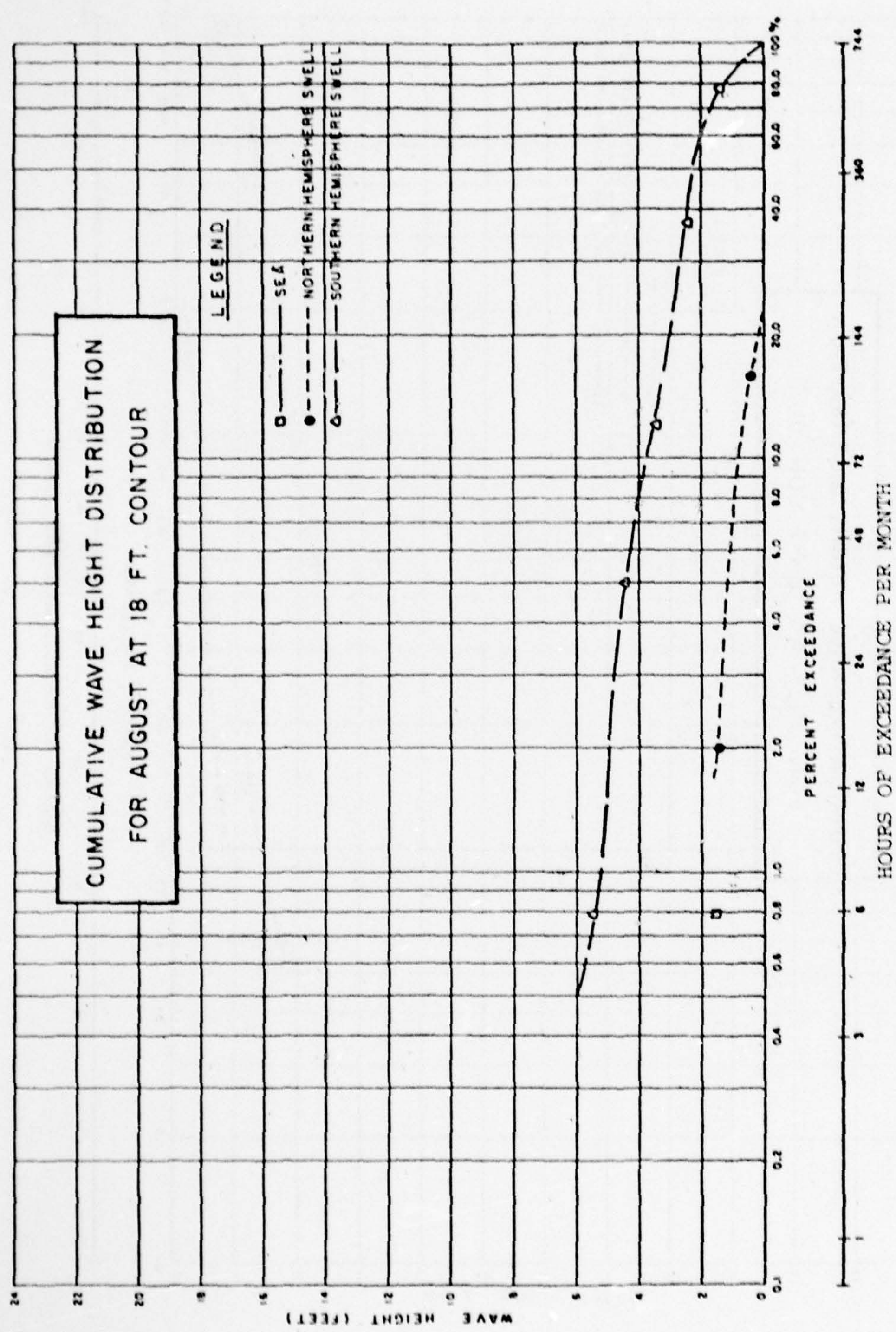

CUMULATIVE WAVE HEIGHT DISTRIBUTION
FOR JUNE AT 18 FT. CONTOUR
LEGEND
SEA
NORTHERN HEMISPHERE SWELL
SOUTHERN HEMISPHERE SWELL
WAVE HEIGHT (FEET)
24
22
20
18
16
14
12
10
8
6
4
2
0
0.1
0.2
0.4
0.6
0.8
1.0
2.0
4.0
6.0
8.0
10.0
20.0
40.0
60.0
80.0
100%
PERCENT EXCEEDANCE
1
3
6
12
24
48
72
144
360
720
HOURS OF EXCEEDANCE PER MONTH

FIGURE C12.



CUMULATIVE WAVE HEIGHT DISTRIBUTION
FOR JULY AT 18 FT. CONTOUR
LEGEND
SEA
NORTHERN HEMISPHERE SWELL
SOUTHERN HEMISPHERE SWEEL
WAVE HEIGHT (FEET)
0
2
4
6
8
10
12
14
16
18
20
22
24
0.1
0.2
0.4
0.6
0.8
1.0
2.0
4.0
6.0
8.0
10.0
20.0
40.0
60.0
80.0
100 %
PERCENT EXCEEDANCE
1
3
6
12
24
48
72
144
360
744
HOURS OF EXCEEDANCE PER MONTH

FIGURE C13.



CUMULATIVE WAVE HEIGHT DISTRIBUTION
FOR AUGUST AT 18 FT. CONTOUR
LEGEND
SEA
NORTHERN HEMISPHERE SWELL
SOUTHERN HEMISPHERE SWELL
WAVE HEIGHT (FEET)
PERCENT EXCEEDANCE
HOURS OF EXCEEDANCE PER MONTH

FIGURE C14.

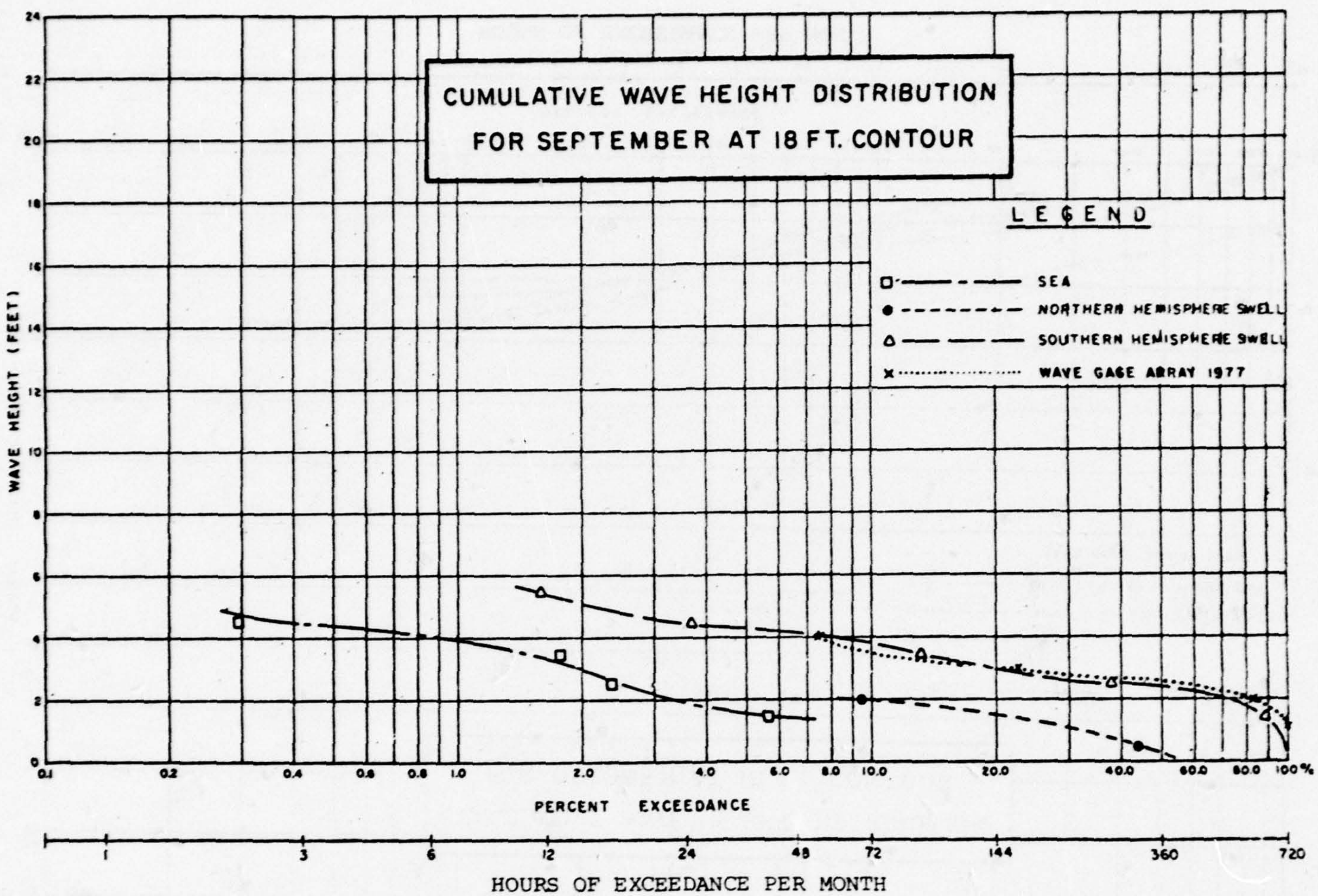

CUMULATIVE WAVE HEIGHT DISTRIBUTION
FOR SEPTEMBER AT 18 FT. CONTOUR
LEGEND
SEA
NORTHERN HEMISPHERE SWELL
SOUTHERN HEMISPHERE SWELL
WAVE GAGE ARRAY 1977
WAVE HEIGHT (FEET)
0
2
4
6
8
10
12
14
16
18
20
22
24
0.1
0.2
0.4
0.6
0.8
1.0
2.0
4.0
6.0
8.0
10.0
20.0
40.0
60.0
80.0
100%
PERCENT EXCEEDANCE
1
3
6
12
24
48
72
144
360
720
HOURS OF EXCEEDANCE PER MONTH

FIGURE C15.

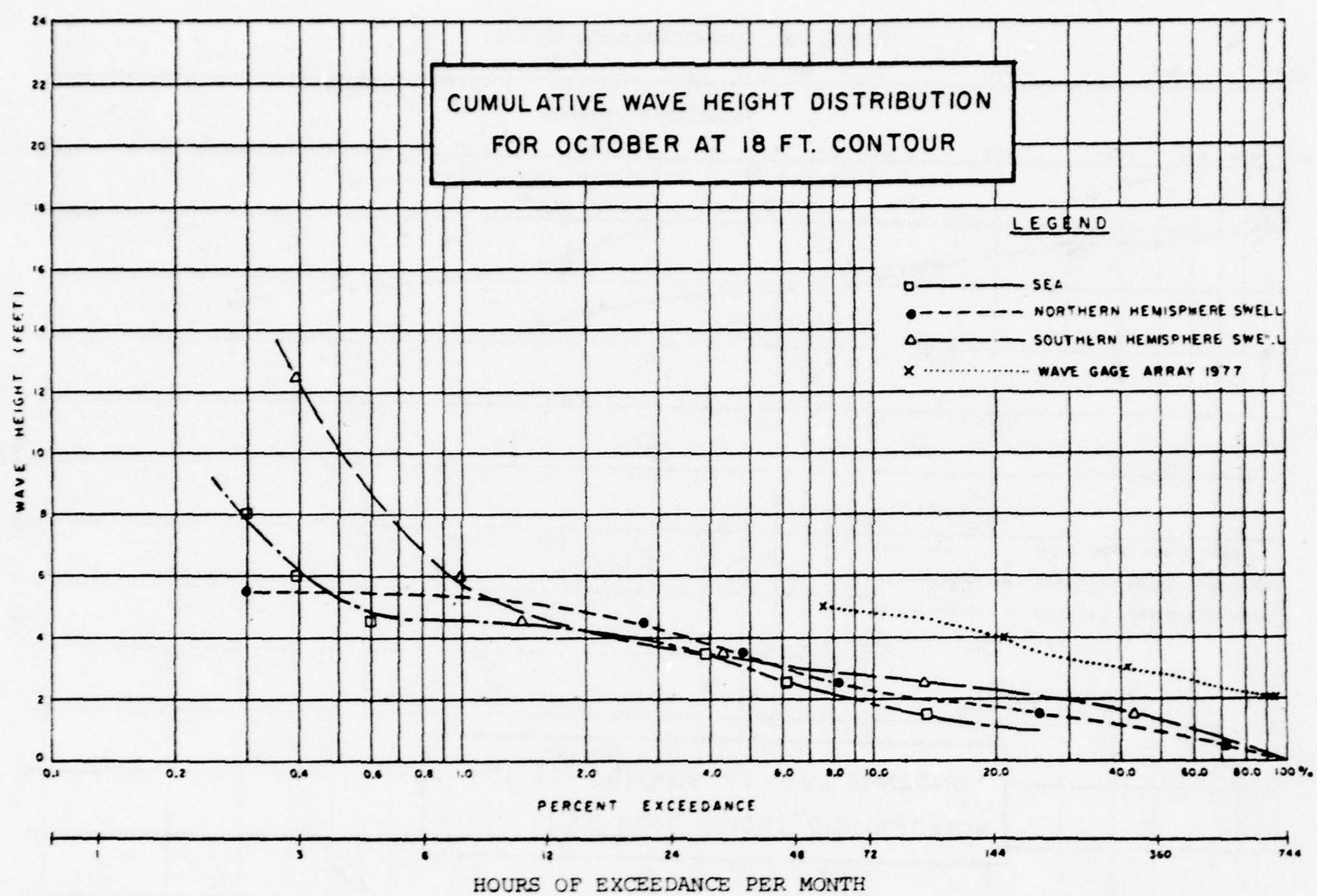

CUMULATIVE WAVE HEIGHT DISTRIBUTION
FOR OCTOBER AT 18 FT. CONTOUR
LEGEND
SEA
NORTHERN HEMISPHERE SWELL
SOUTHERN HEMISPHERE SWELL
WAVE GAGE ARRAY 1977
WAVE HEIGHT (FEET)
0
2
4
6
8
10
12
14
16
18
20
22
24
0.1
0.2
0.4
0.6
0.8
1.0
2.0
4.0
6.0
8.0
10.0
20.0
40.0
60.0
80.0
100%
PERCENT EXCEEDANCE
1
3
6
12
24
48
72
144
360
744
HOURS OF EXCEEDANCE PER MONTH

FIGURE C16.

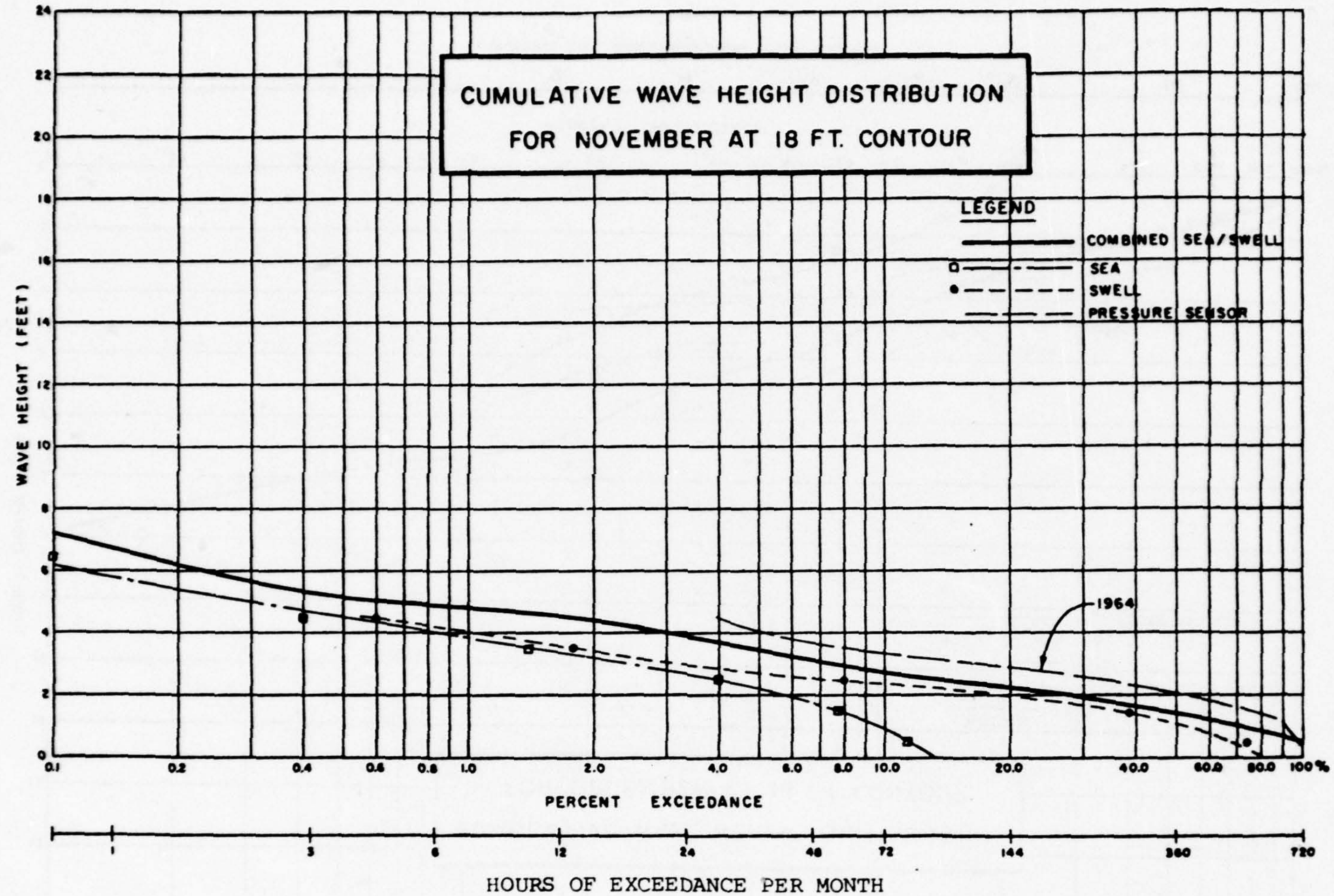

CUMULATIVE WAVE HEIGHT DISTRIBUTION
FOR NOVEMBER AT 18 FT. CONTOUR
LEGEND
COMBINED SEA/SWELL
SEA
SWELL
PRESSURE SENSOR
1964
WAVE HEIGHT (FEET)
0
2
4
6
8
10
12
14
16
18
20
22
24
0.1
0.2
0.4
0.6
0.8
1.0
2.0
4.0
6.0
8.0
10.0
20.0
40.0
60.0
80.0
100%
PERCENT EXCEEDANCE
1
3
6
12
24
48
72
144
360
720
HOURS OF EXCEEDANCE PER MONTH

FIGURE C17.

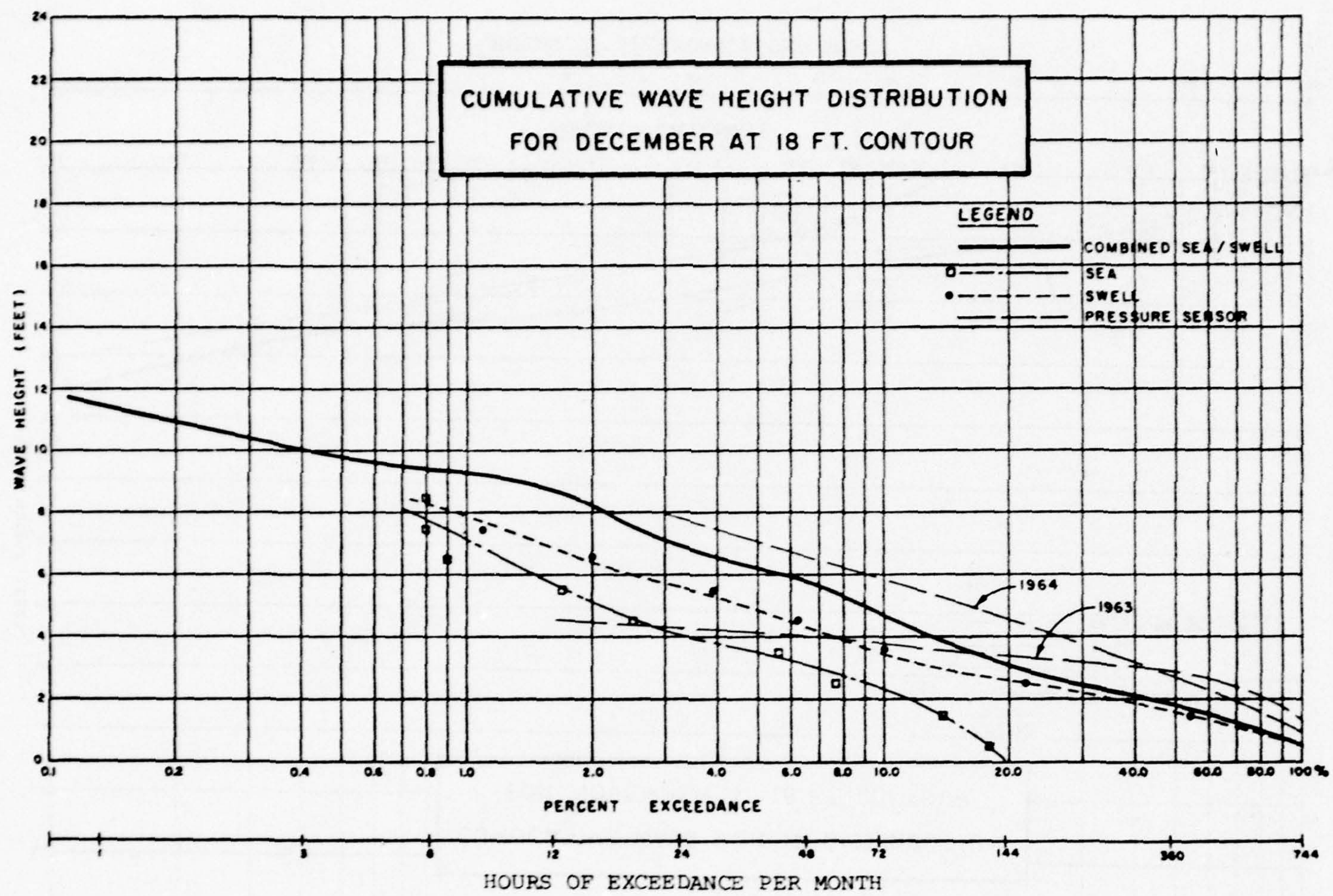

CUMULATIVE WAVE HEIGHT DISTRIBUTION
FOR DECEMBER AT 18 FT. CONTOUR
LEGEND
COMBINED SEA / SWELL
SEA
SWELL
PRESSURE SENSOR
1964
1963
WAVE HEIGHT (FEET)
0
2
4
6
8
10
12
14
16
18
20
22
24
0.1
0.2
0.4
0.6
0.8
1.0
2.0
4.0
6.0
8.0
10.0
20.0
40.0
60.0
80.0
100%
PERCENT EXCEEDANCE
1
3
6
12
24
48
72
144
360
744
HOURS OF EXCEEDANCE PER MONTH

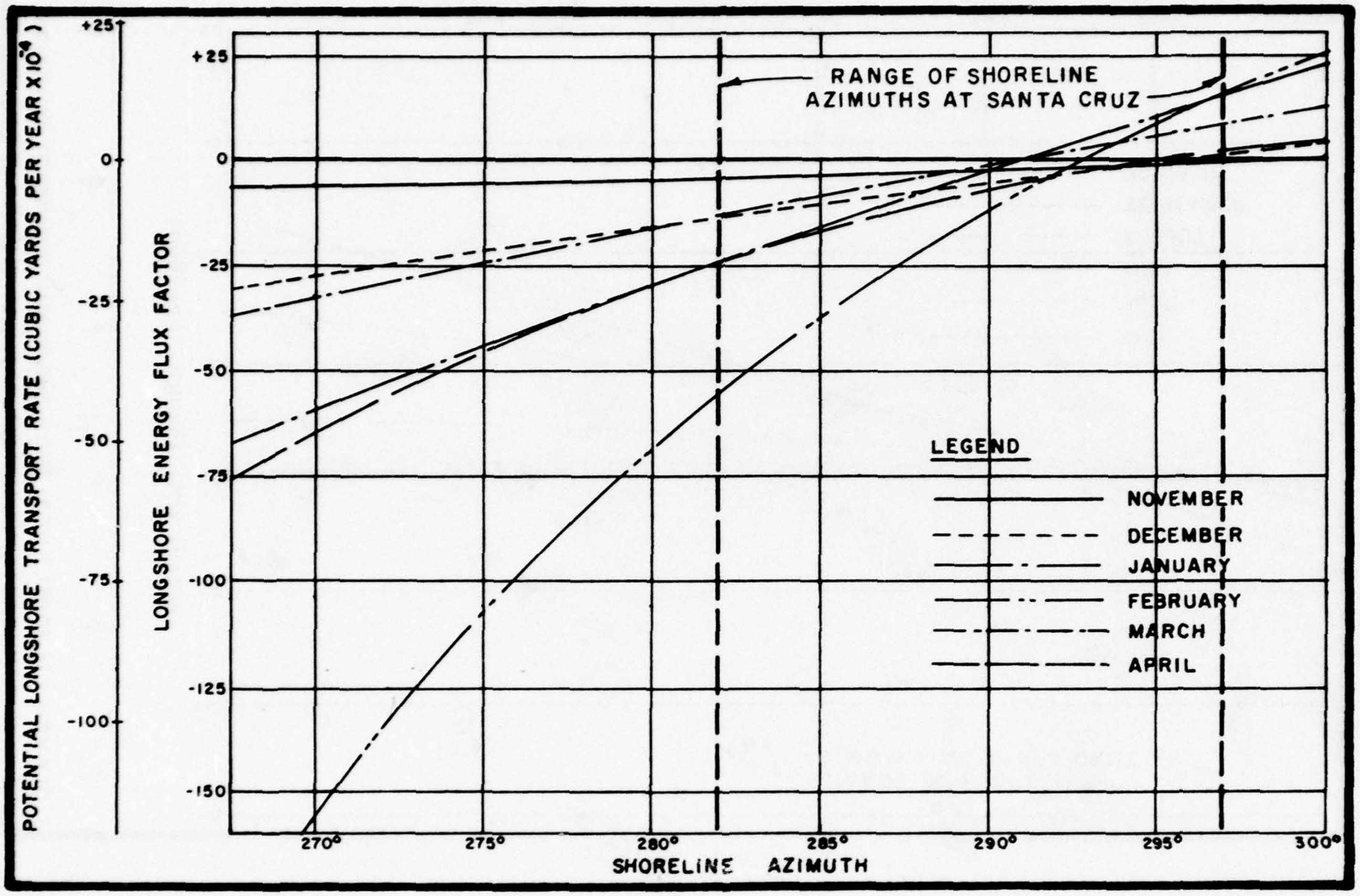
FIGURE C18.

C.13 Measured wave data from the slope-array gage are also plotted for September and October 1977 and February, March, and April 1978. The maximum daily significant height is presented for a comparison of the predicted and measured climates. The shapes of the cumulative distributions of the predicted and measured data are in general agreement; however, the wave gage indicates that the occurence of low waves is under-predicted in the hindcast wave climate. Several factors can explain the discrepancy. Low-height waves from several sources may arrive simultaneously at the gage, all of which may not have been considered in the hindcasts. The wave gage measures water pressure and transforms the data into a surface wave spectrum. Several theoretical assumptions must be made to convert the pressure distributions into a description of surface waves. Some of these assumptions are not necessarily valid in shallow water near the breaker line. The sampling procedure also may bias the results because most measurements of the spectra were taken during daylight hours when the daily breeze may generate small waves. A rigorous analysis of the wave-gage data and the predicted data is beyond the scope of this study, but such an analysis should be made before several years of data are obtained.

LITTORAL TRANSPORT

C.14 Energy-Flux Calculations. The potential for waves to transport littoral drift along a beach can be estimated by applying an empirical factor relating littoral transport to the longshore component of wave-energy flux. The longshore component of wave-energy flux is primarily a function of the angle of wave approach relative to the depth contours and of the square of the wave height. Appendix 2 describes the method used to transform the incident deep-water wave climate to a breaking-wave climate at the project site. Basically, the method applies refraction and shoaling coefficients and change of direction of propagation to the deep-water wave to determine the wave climate in 18 feet of water offshore from the harbor. By assuming straight and parallel contours seaward and shoreward of this depth, additional refraction and shoaling are calculated until the wave satisfies empirical wave-breaking criteria. The assumption of a constant shoreline orientation, which in reality varies spacially and temporally, renders the method an approximation.

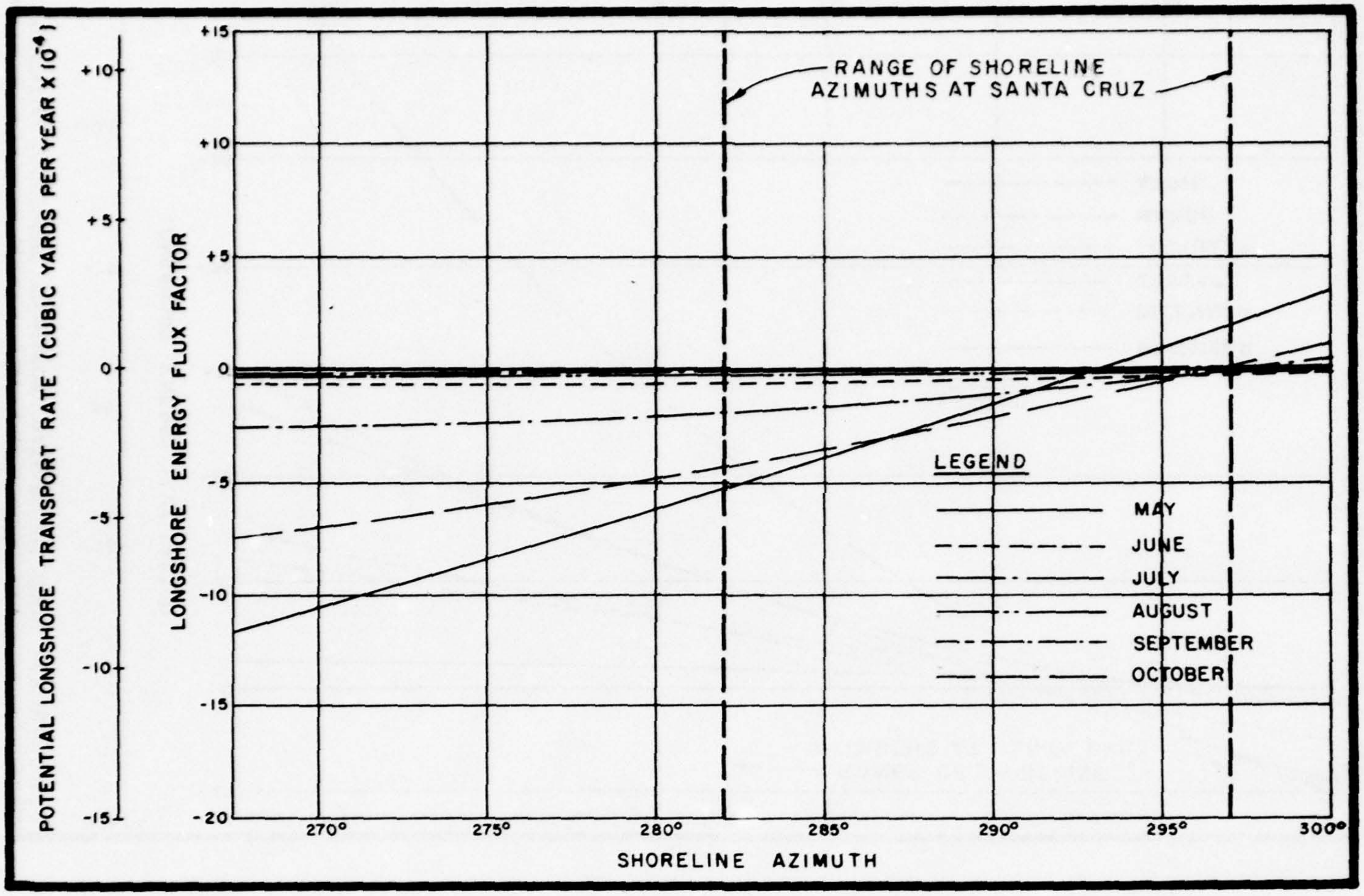
C.15 Wave-energy flux was calculated for each data point in the sea and swell components of the wave climate for each month. Figures C19, C20, and C21 show variation of longshore energy flux as a function of shoreline alignment between azimuths of 270° and 300°. A negative sign denotes an eastward transport and a positive sign denotes a westward transport. The calculations show the degree of sensitivity of the littoral-transport rate as a function of swell and assumed shoreline azimuth for each month. The results given in figures C19, 20 and 21 show that the calculations are not highly sensitive to the range of shoreline azimuths characteristic of the harbor area. The longshore energy flux of the Northern Hemisphere swell is an order of magnitude greater than that of the Southern Hemisphere swell. The plots also show how the waves are attempting to realign the beach toward zero transport each season. During the summer, the Southern Hemisphere swell is attempting to align the beach on a 283° to 288° azimuth, whereas during the winter the Northern Hemisphere swell is attempting to align the beach on an azimuth between 290° and 296°.



NOVEMBER THRU APRIL

LONGSHORE ENERGY FLUX FACTOR VS. SHORELINE ALIGNMENT
NORTHERN HEMISPHERE SWELL

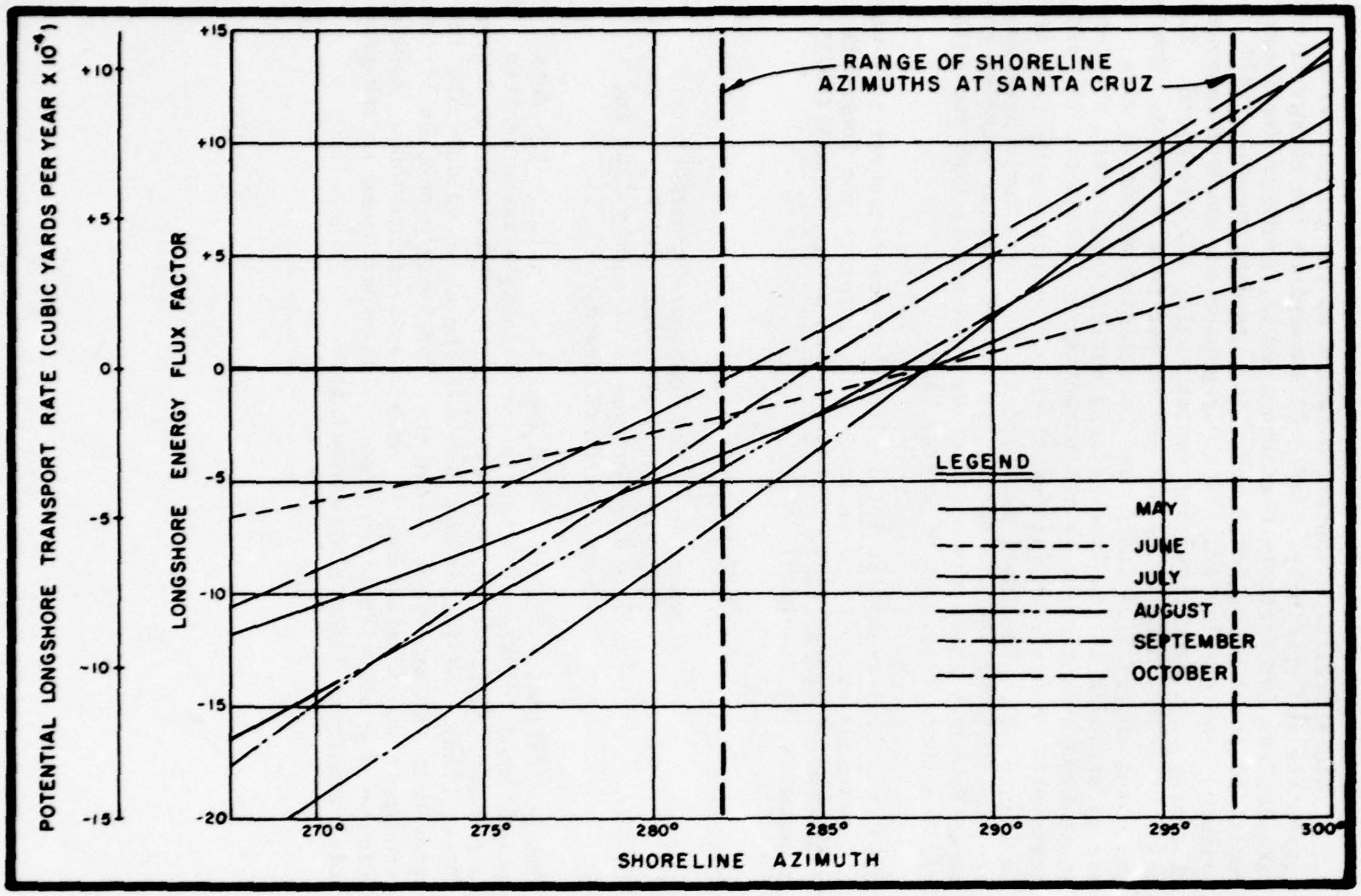
FIGURE C19



MAY THRU OCTOBER

LONGSHORE ENERGY FLUX FACTOR VS. SHORELINE ALIGNMENT
NORTHERN HEMISPHERE SWELL

FIGURE [illegible].



MAY THRU OCTOBER

LONGSHORE ENERGY FLUX FACTOR VS. SHORELINE ALIGNMENT
SOUTHERN HEMISPHERE SWELL

FIGURE C21.

C.16 The shoreline orientation varies at the project site seasonally and spacially. Table C3 summarizes the energy-flux calculations for Northern Hemisphere swell, Southern Hemisphere swell and local seas, for a shoreline trending toward a 287° azimuth. The total energy flux, longshore and onshore components of energy flux, and resultant vector angle, are presented. The units of energy are ft-lbs per second per foot of beach. Whereas the total energy flux indicates the relative amount of wave energy arriving at the breaker zone during each month, the longshore component represents the potential for longshore littoral transport. Monthly variations of the longshore energy flux are plotted in figure C22. The local seas and the Northern Hemisphere swell are the dominant factors during the winter between December and May. February shows the most active wave-energy flux and greatest longshore component of energy flux.

C.17 Littoral-Transport Rate. The littoral-transport rate may be estimated by applying an empirical factor to the longshore component of energy flux. The Shore Protection Manual (1973) recommends the equation:

$$Q = 7.5 \times 10^3 P_{1s};$$

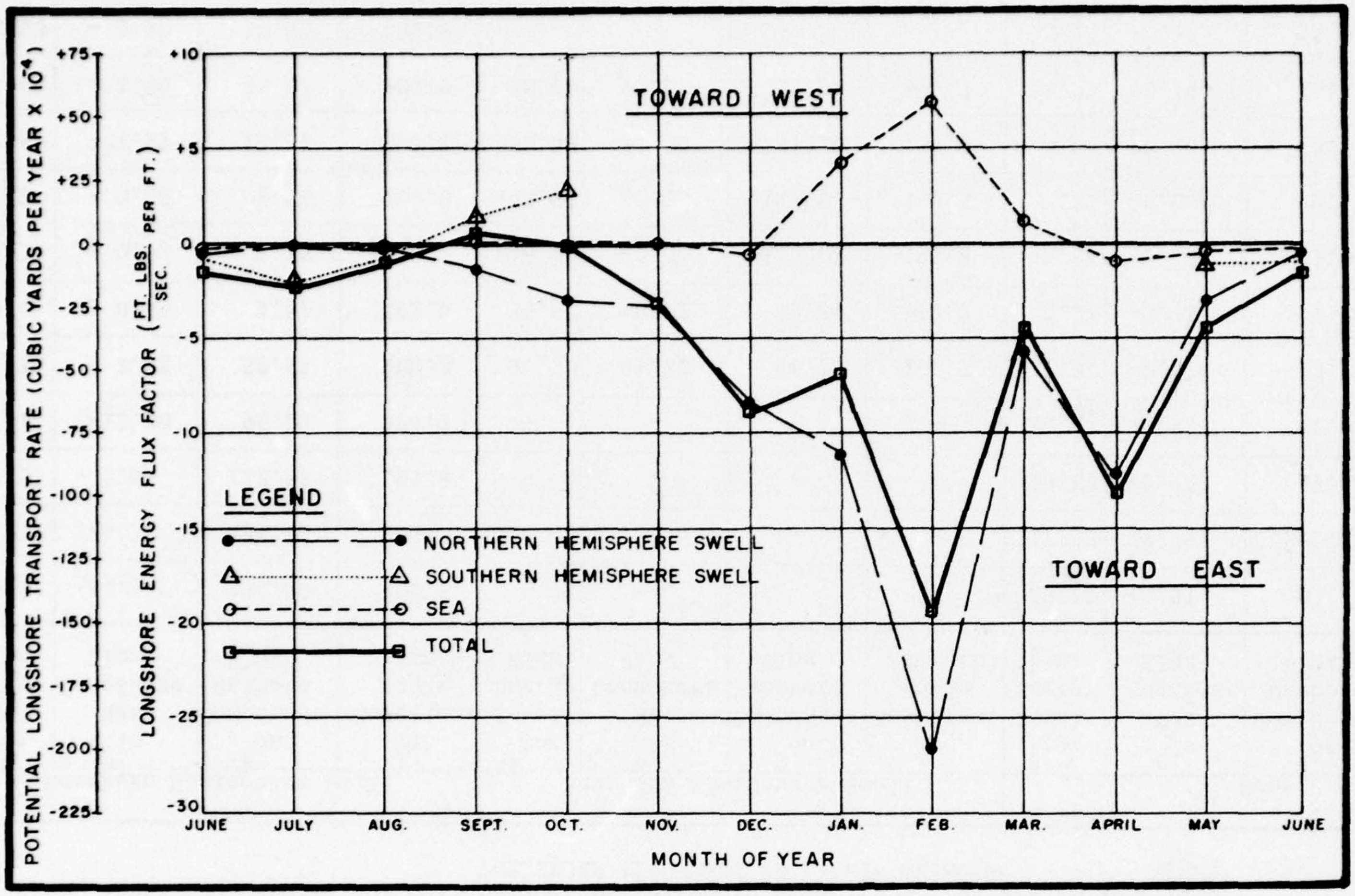
where Q is the longshore transport rate in cubic yards per year; and P_{1s} is the longshore component of energy flux in ft-lbs/sec/ft of beach.

The coefficient 7.5×10^3 is an empirical factor. The data points used to determine the factor display a large scatter. In a previous publication, Shore Protection, Planning, and Design TR-4 (1966), the factor was one-half the above value. The increase in the empirical factor therefore nearly doubles littoral-transport rates determined by previous investigations. Application of the above equation predicts an upper bound to the potential longshore littoral-transport rate.

TABLE C3. SUMMARY OF WAVE ENERGY FLUX AT SANTA CRUZ (ft-lb/sec per ft of beach) FOR SHORE ALIGNMENT ON A 287° AZIMUTH

Month	Northern Hemisphere Swell				Southern Hemisphere Swell				Seas			
	P_{tot} Total Energy Flux	$P_{\ell s}$ Net Longshore Flux	P_{os} Component Onshore Flux	ϕ_r Resultant Angle Azimuth	P_{tot} Total Energy Flux	$P_{\ell s}$ Net Longshore Flux	P_{os} Component Onshore Flux	ϕ_r Resultant Angle Azimuth	P_{tot} Total Energy Flux	$P_{\ell s}$ Net Longshore Flux	P_{os} Component Onshore Flux	ϕ_r Resultant Angle Azimuth
Jan.	195.42	-11.16	194.40	200.3	-	-	-	-	88.10	+4.21	87.17	194.2
Feb.	301.34	-76.77	299.92	202.1	-	-	-	-	217.53	+7.43	216.56	195.0
Mar.	113.22	- 5.63	112.57	199.9	-	-	-	-	49.63	+1.14	49.11	195.7
Apr.	97.14	-12.30	96.22	204.3	-	-	-	-	43.17	-0.93	42.92	198.2
May	50.78	- 2.92	50.57	200.3	96.78	-1.22	96.68	197.7	17.31	-0.47	17.20	198.6
Jun.	5.43	- 0.45	5.41	201.8	59.67	-0.72	59.64	197.7	2.27	-0.31	2.25	204.9
Jul.	1.35	- 0.09	1.35	200.9	146.20	-2.06	146.07	197.8	1.38	-0.10	1.37	201.3
Aug.	1.87	- 0.16	1.86	202.0	117.07	-0.83	116.92	197.4	.44	-0.06	0.43	204.5
Sep.	17.43	- 1.27	17.37	201.2	132.39	+1.59	132.18	196.3	6.04	+0.10	5.99	196.0
Oct.	35.69	- 3.09	35.52	202.0	70.60	+2.87	70.46	194.7	14.26	+0.19	14.13	196.2
Nov.	32.80	- 3.20	32.60	202.6	-	-	-	-	5.14	+0.17	5.01	195.1
Dec.	94.06	- 8.53	93.47	202.2	-	-	-	-	23.45	-0.51	23.21	198.2

(+) to the west
(-) to the east



LONGSHORE ENERGY FLUX FACTOR VS. MONTH

FIGUR 22.

C.18 Results. Potential littoral-transport rates may be estimated by applying the above equation to plots of longshore energy flux. The ordinates of figures C19 through C21 show the potential for littoral transport as well as longshore wave-energy flux. Table C4 summarizes the monthly distribution of total energy flux, the longshore component of energy flux, and the potential longshore-transport rate. This table was derived from table C3, by summarizing the Northern and Southern Hemisphere swells and local seas to derive a composite net monthly littoral-transport rate. The potential net littoral-transport rate is 488,000 cubic yards per year to the east. Approximately 80 percent of this occurs during the winter, between December and April.

C.19 Reversals in littoral transport, or the tendency for littoral drift to move upcoast contrary to the net direction of movement, are summarized in table C5. This table presents the net westward and eastward energy-flux factors. The Northern Hemisphere swell results primarily in uni-directional eastward transport because the primary source of waves is from the northwest. The eastward transport is an order of magnitude greater than the westward transport. Seas and Southern Hemisphere swell are more balanced. During January, February, and March the seas tend to cause a reversal, similar to that found for the Northern Hemisphere; however, this reversal is of weaker magnitude than that for the predominant Northern Hemisphere swell.

C.20 Data from the array gage system have been used to calculate the energy flux for October 1977 and February 1978. The results are presented in figures C23 and C24. The net littoral-transport rate for October 1977 was 2600 cubic yards toward the east; however, the transport was toward the west for most of the month. During the month of February, the wave-gage analysis predicted the potential littoral-transport rate to be 35,000 cubic yards toward the east with only a small reversal. This is considerably lower than the estimated 145,000 cubic yards for a typical February. Differences lie in the short period of wave measurements and in orientation of shoreline selected for analysis. In the wave gage analysis a shoreline oriented due west was assumed, whereas in the calculation a shoreline oriented 287° true north azimuth was assumed. Referring to figure C19, one would expect this 17° difference in assumed shoreline orientation

TABLE C4

TOTAL NET MONTHLY LITTORAL-TRANSPORT RATES			
Month	Total Energy Flux ft-lb/sec/ft	Longshore Component ft-lb/sec/ft	Littoral-Transport Rate cu.yd./yr.
Jan.	284	- 6.95	- 52,125
Feb.	519	-19.34	-145,050
March	163	- 4.49	- 33,675
April	140	-13.23	- 99,675
May	164	- 4.61	- 34,575
June	67	- 1.48	- 11,400
July	149	- 2.25	- 16,875
Aug.	119	- 1.05	- 7,875
Sept.	156	+ 0.42	+ 3,150
Oct.	121	- 0.03	- 225
Nov.	38	- 3.03	- 22,725
Dec.	118	- 9.04	- 67,800
Total	2,038	-65.08	-488,100

TABLE C5

NET WESTERLY AND EASTERLY ENERGY-FLUX FACTORS (ft-lb per sec per ft)						
	Northern Hemisphere Swell		Southern Hemisphere Swell		Sea	
Month	Westward	Eastward	Westward	Eastward	Westward	Eastward
Jan.	2.07	13.86	-	-	6.89	2.68
Feb.	0.08	26.86	-	-	11.41	3.98
March	2.51	8.14	-	-	3.36	2.21
April	0.00	12.30	-	-	1.42	2.35
May	0.64	3.55	1.20	2.43	0.52	1.00
June	0.00	0.45	0.64	1.36	0.01	0.32
July	0.00	0.09	1.58	3.64	0.00	0.10
Aug.	0.00	0.16	2.10	2.93	0.00	0.06
Sept.	0.00	1.27	3.87	2.28	0.40	0.30
Oct.	0.00	3.09	3.29	0.42	0.92	0.73
Nov.	0.00	3.20	-	-	0.52	0.35
Dec.	0.58	9.12	-	-	1.00	1.51

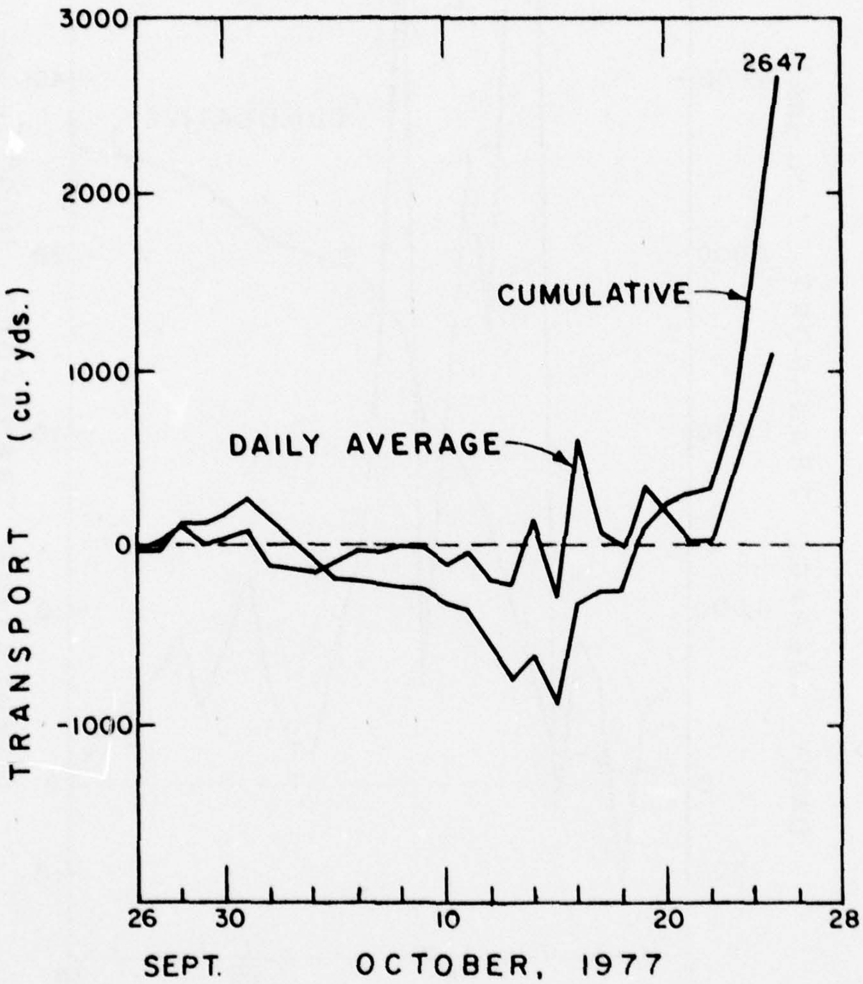


FIGURE C23: LITTORAL TRANSPORT RATES PREDICTED FROM ANALYSIS OF WAVE-GAGE ARRAY DATA, OCTOBER 1977.

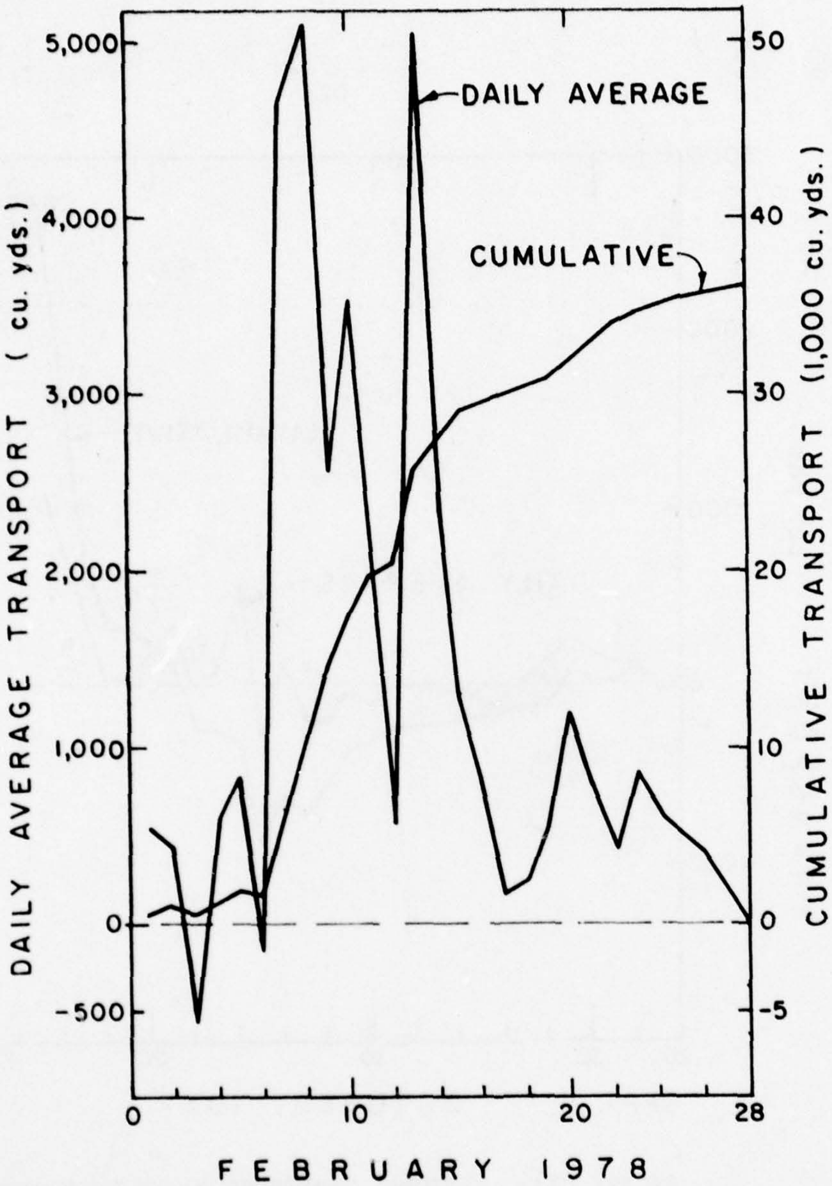


FIGURE C24: LITTORAL TRANSPORT RATES PREDICTED FROM ANALYSIS OF WAVE-GAGE ARRAY DATA, FEBRUARY 1978.

to yield significantly different results. If the results of the wave-gage analysis prove more representative, the littoral transport rate estimated from the derived wave climate should be considerably lower than that estimated in this study. The derived wave climate includes a considerable contribution of energy flux by waves greater than seven feet in height. Furthermore, the littoral transport analysis did not assume Airy theory for wave shoaling to the breaker point as does the wave-gage analysis. A more complex set of wave transformations was used in the analysis of the derived wave climate to account for the effects of non-linear shoaling and refraction. Additional years of data acquisition are required for a more meaningful comparison or re-estimation of littoral transport rates.

BATHYMETRIC CHANGES

C.21 The seasonal and long-term changes in bathymetry are discussed in this section to give insight into the shoaling process. The long-term growth of the fillet formed on the west side of the west jetty and the seasonal shoal development in the harbor entrance channel are discussed.

C.22 Data. The analysis of beach and contour changes was based primarily on hydrographic surveys which were taken along various range lines between the San Lorenzo River and Black Point. Table C6 summarizes the survey data that were available between 1953 and 1978. The shoaling patterns were analyzed by comparing condition surveys and hydrographic surveys in the vicinity of the jetties. In later years these surveys were taken on a monthly basis, permitting an investigation of seasonal changes in beach alignment, contours, and shoaling patterns. Aerial photographs were used as supplemental tools to interpolate and extrapolate measured shoreline data.

C.23 Shoreline and Bathymetry Changes. Plates 1 through 5 show the evolution of the mean lower low water shoreline and the 6-, 12- and 18-foot depth contours as determined from the survey data. Figure C25 shows the shoreline growth that occurred at three profiles west of the west jetty. By 1977, the shoreline 200 feet west of the west jetty had aggraded nearly 500 feet seaward of the pre-project shoreline. The shoreline growth was

Table C6

SANTA CRUZ HARBOR, CALIFORNIA
LITTORAL PROCESSES STUDY

REFERENCE SURVEYS

February, 1978

	1953	1954	1955	1956	1957	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975	1976	1977	1978	
JAN.		BP	BP									C BE C	BD				C	BD	BD		BD	RH	C BE	C	C	H BP	JAN.
FEB.		BP									C BE BE	BE BE			C C	BD					DD 109	BD RH	C BE	C	C	H	FEB.
MAR.		BP	BP BP	XGP							DC	R BE	BE	BE		C	BD BD 79	BD 94	BD BD		AD	RH 60	C BE	98	C	H	MAR.
APR.		BP		XGP							BD		BD	C		BD 60	AD	AD	AD	90 BD	C	RH	BD AD 91		BP	H	APR.
MAY		BP		XGP							BE			BD	BD			AD 11	BD AD 108	AD	C	RH BE	C		147	162	MAY
JUNE		BP	XGP								DD		70	BD	AD 57	AD				DA		RH BE		C	C		JUNE
JULY		BP	XGP	XGP									BD	AD C 34							RH	WES Operating Periods		C	C		JULY
AUG.		BP	XGP	XGP							BD AD		AD BE	BE							RH			C BP	H BP		AUG.
SEPT.		BP	XGP							C	BE DD BP		BE		BE						RH			C	H		SEPT.
OCT.			XGP XGP	XGP		BE			C	DC BE	AD AD AD	BE BE	BE	C BE			C				RH		C	C	H		OCT.
NOV.			XGP XGP XGP	XGP XGP		BE			BP		BE C C	BE BD	BD			C					RH			C	H		NOV.
DEC.	BP			XGP						DC BE	BE									BD	RH		C	C	H		DEC.

H HYDROGRAPHIC SURVEY
BP BEACH PROFILE
XGP EXPERIMENTAL GROIN PROFILE
BE BEACH EROSION
C CONDITION SURVEY
DC DURING CONSTRUCTION

70 | INDICATES ANNUAL MAINTENANCE DREDGING PERIOD AND ASSOCIATED "PAY YARDAGE" IN 1000 CU. YARDS
DD DURING DREDGING
AD AFTER DREDGING
BD BEFORE DREDGING
RH REPETITIVE HYDRO
R RECONNAISSANCE

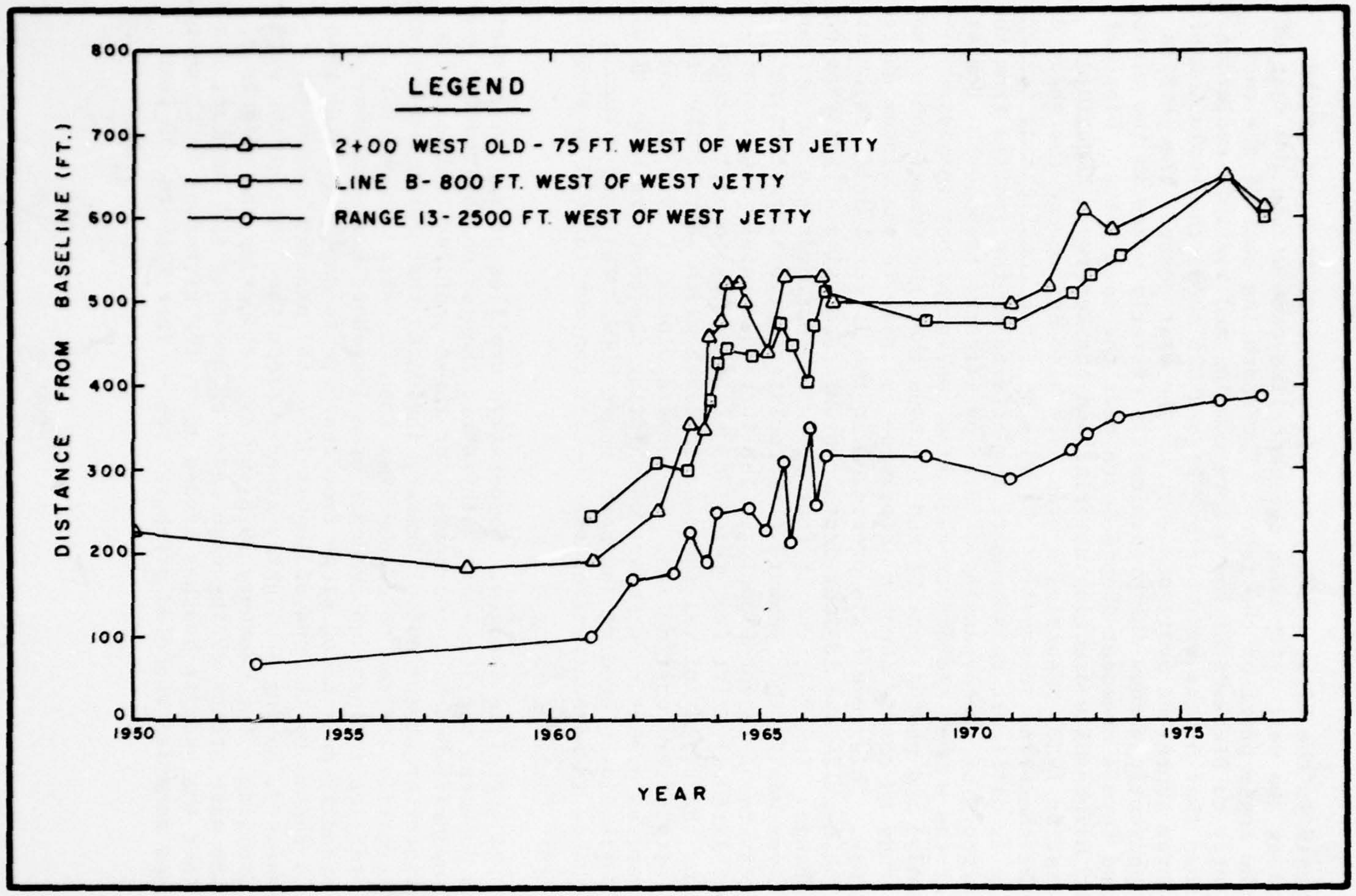


FIGURE C25. WEST BEACH SHORELINE CHANGES

rapid in the first three years after construction and ceased along the west jetty when the shoreline came within 100 feet of the angle point of the jetty. The shoreline east of the east jetty to Black Point had a more complex and variable evolution than that of the west beach. Figure C26 shows the evolution of three shoreline positions east of the east jetty. The jetties apparently shadow the predominantly westerly waves in the winter and form a crescent-shaped beach near the east jetty. Disposal of maintenance-dredging material and the Waterways Experiment Station (WES) bypassing system have also influenced the shape of the shoreline seasonally. Because of these complicating factors, it is difficult to segregate the effects of the jetties from the effects of the bypassing efforts on littoral transport. Comparison of the summer and winter shoreline configurations in plates 1 and 5 and photographs C1 and C2 shows that the summer shoreline tends to rotate about 6° clockwise relative to the winter shoreline. This result was predicted in the littoral-drift calculation. The 6-, 12- and 18-foot depth contours show similar patterns of change. In 1963, the 18-foot depth contour near the San Lorenzo River was farther seaward than in later surveys, reflecting results of recent flooding. The increased supply and trapping of littoral drift from this 1963 flooding may in part account for the rapid initial increase in beach width west of the west jetty after construction. The severe winter of 1978 did not have as dramatic an impact on shifting the contours below MLLW as it did on eroding the beach above MLLW. The most dramatic change is advancement of the 18-foot contour in 1978 as shown in plate 4.

C.24 Profile Changes. Comparative profiles presented in plate 6 document the long-term bathymetric changes on the updrift and downdrift beaches. Comparison of these profiles, taken in the winter or at the end of summer, indicates that depth changes are primarily limited to depths less than 20 feet. The winter profiles indicate no evidence of a longshore bar, but show a steadily decreasing slope from the berm to depths greater than 20 feet. By the end of August 1977, the profile had moved seaward, having a slightly steeper slope than that of the winter profiles. The February 1978 survey, reflecting the effects of the wave attack during high tides of December and January, shows that the updrift beaches eroded above the five-foot depth contour and material accreted offshore from -5 feet MLLW to -20 feet

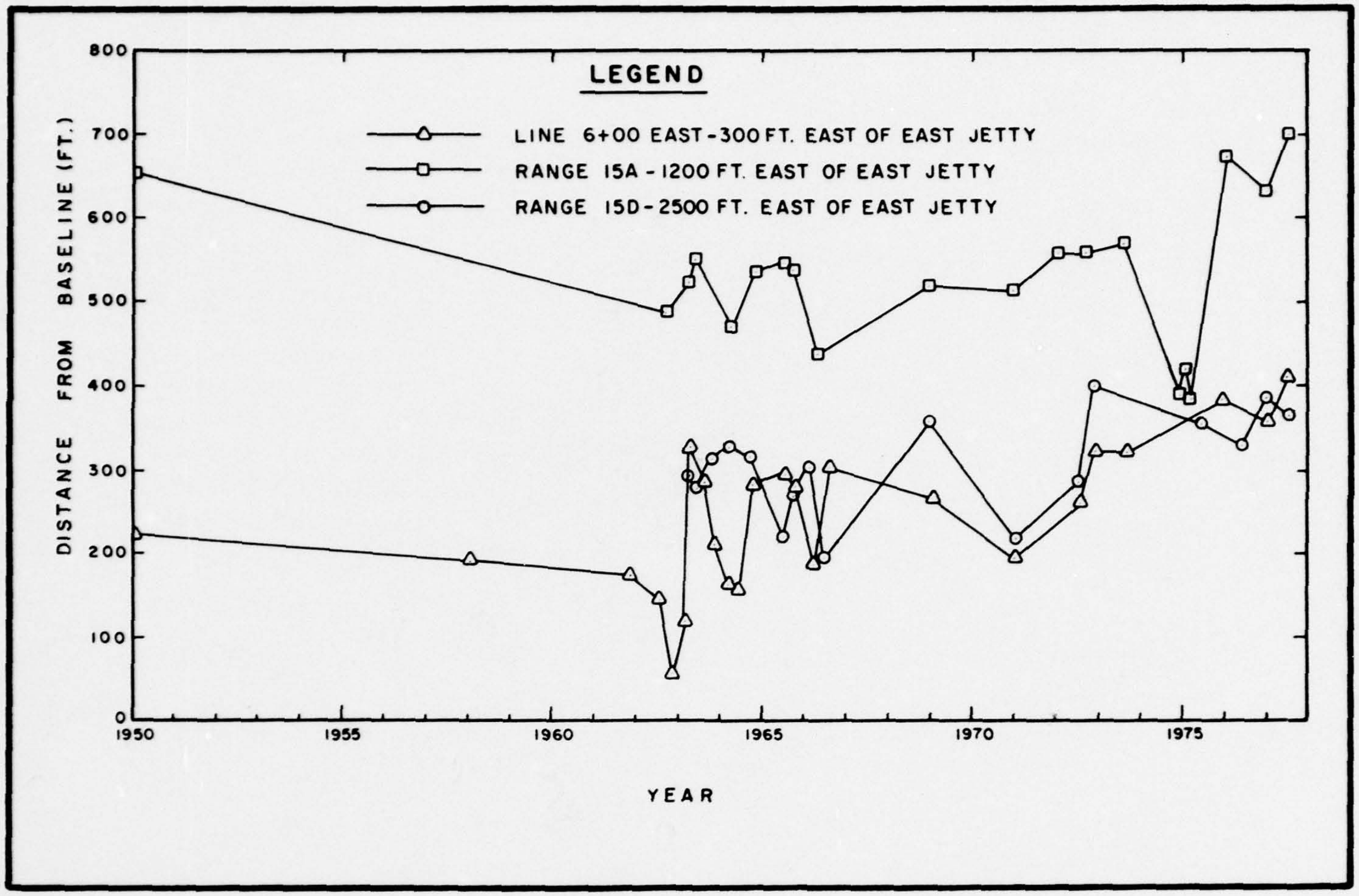


FIGURE C26.   EAST BEACH SHORELINE CHANGES

Photograph C1: Summer Shoreline Configuration, October 20, 1977

Photograph C2: Winter Shoreline Configuration, January 20, 1978

MLLW. This change in profile normally occurs every winter; however, in 1978 it was more severe than in other recent winters of record. The downdrift profiles eroded five feet deeper, but material was deposited within the 20-foot depth contour.

C.25 The profiles show that the nearshore slope averages 1 on 30 at the west jetty, Range 2+00. Bathymetric profiles off the head of the west jetty were constructed from condition surveys and are compared in figure C27, which illustrates the long-term stability of the salient offshore of the west jetty. The winter profile seaward of the doglegged west jetty is steeper than 1 on 15. This slope intersects the jetty at about -2 feet MLLW. Seasonal profile changes are more dramatic, as shown in figure C28. The salient intersects the jetty at about -7 feet by the end of summer (October), but shoals during the winter. The two figures indicate a relative long-term stability achieved prior to January 1969. The lack of data prior to this time prevents a more detailed determination of when this salient became saturated.

C.26 Channel Shoaling. The shoaling patterns of the entrance channel have been documented by condition surveys taken monthly over the past five years, pre- and post-dredge surveys, and repetitive hydrographic surveys. Plates 7 and 8 show the monthly progression of the MLLW and the 10-foot depth contours in 1975-1976 and 1976-1977 winter seasons, respectively. The patterns developed in similar fashion each year, with a tip shoal developing and then extending from the head of the west jetty across the channel toward the head of the east jetty. By January, the 10-foot depth contour had closed across the channel and the beach to the east had receded. The influx of material into the channel during the winter appears to result primarily from sand bypassing the head of the west jetty. Support for this hypothesis is evident in plate 8 from the advance of the 10-foot depth contour east of the west jetty from September to January and the recession of the 10-foot depth contour off the east jetty during the same period. Recalling the clockwise shift in beach alignment from summer to winter and the recession of the beach at the east jetty in the winter, the primary source of shoaling appears to be from a net eastward littoral transport. Littoral drift bypassing the west jetty forms a tip shoal, allowing part of the material to bypass the channel to the downdrift cell. The growth of the bar across the channel between the two jetty heads is shown in figure C29 by cross-sections taken monthly during 1976-1977.

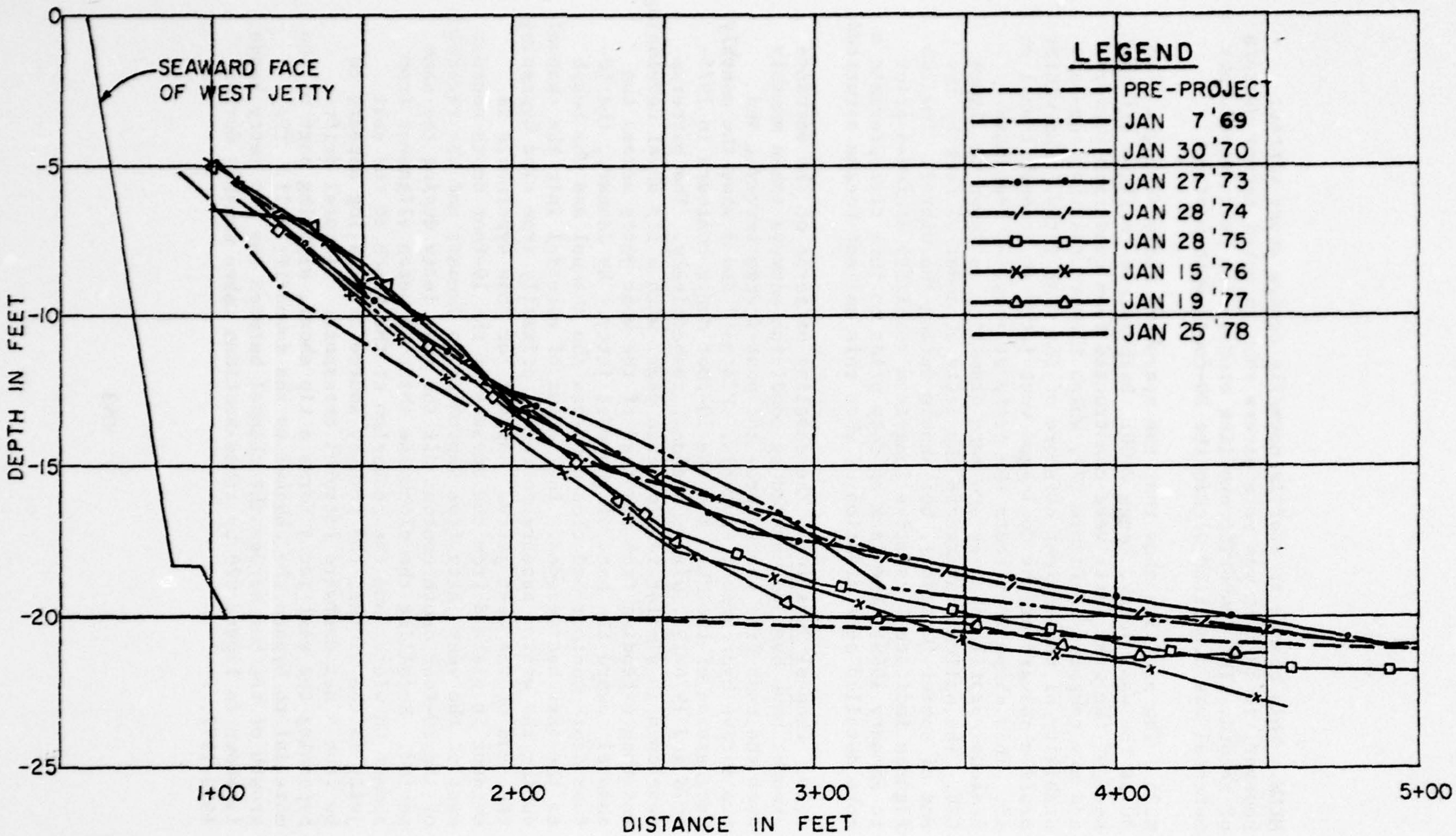


Figure C27 PROFILES SEAWARD OF WEST JETTY
Pre-Construction - 1977
(Station 21+00)

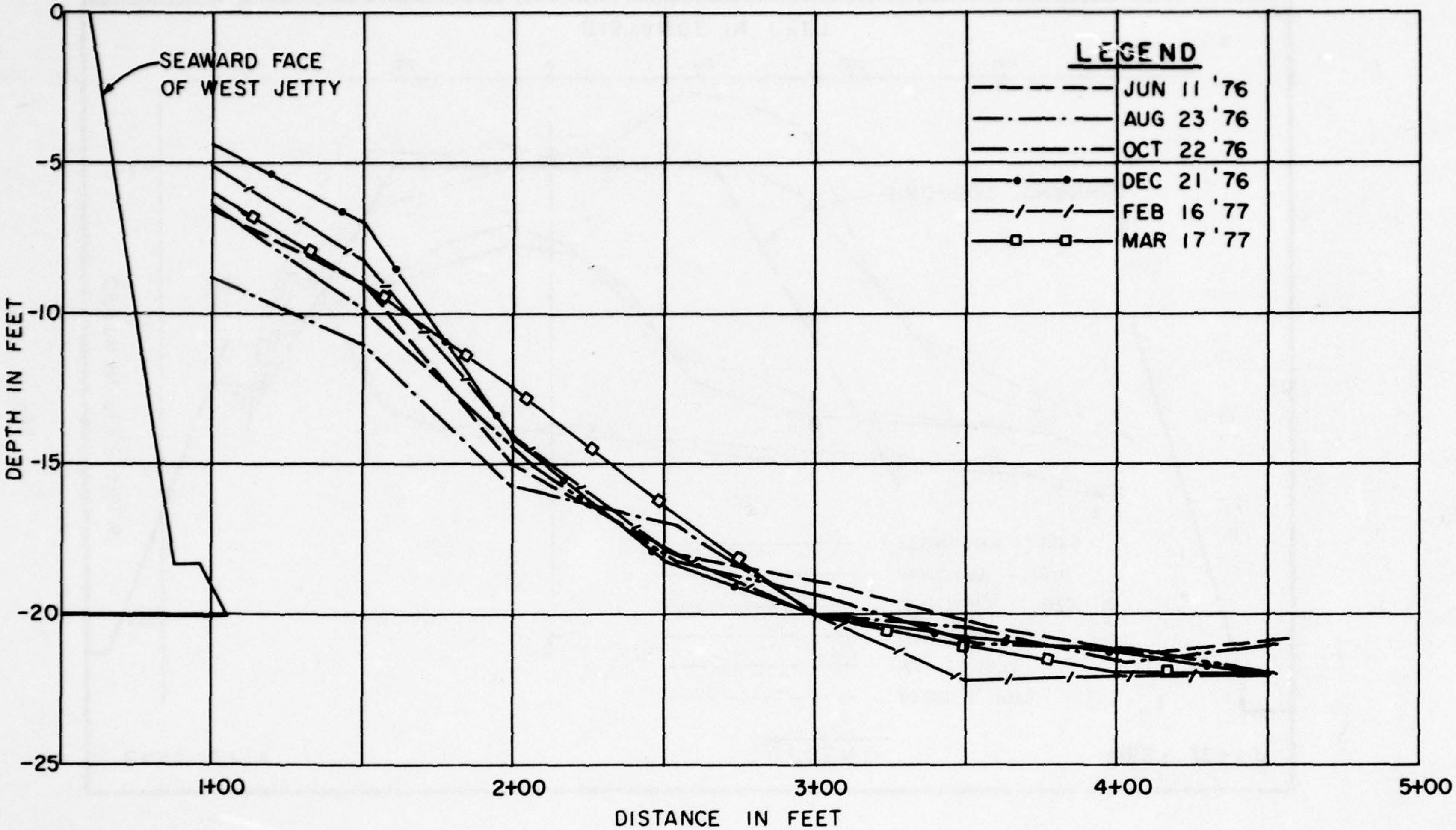


Figure C28 PROFILES SEAWARD OF THE WEST JETTY
Seasonal Variations
(Station 21+00)

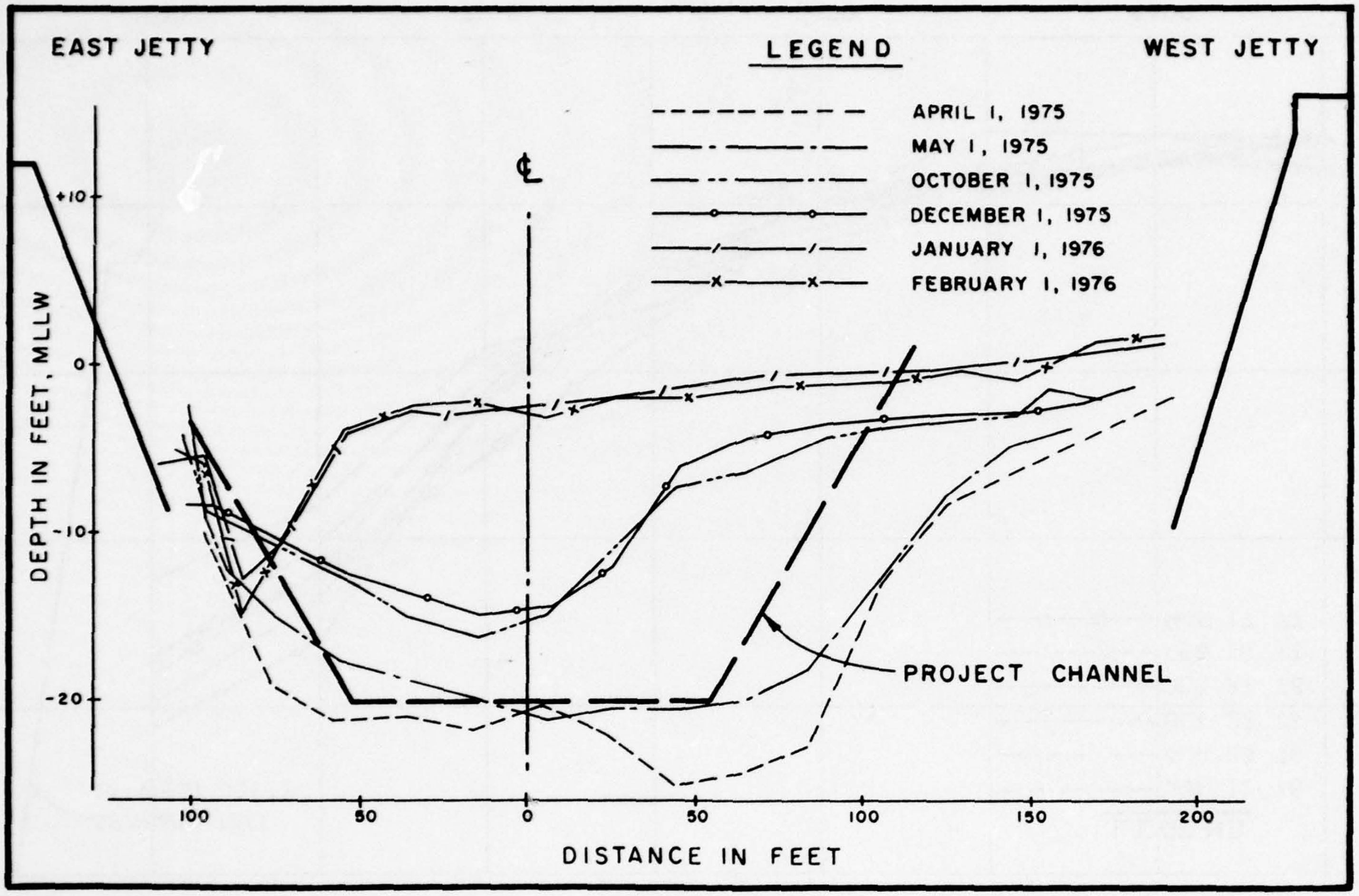


FIGURE C29. CROSS-SECTION BETWEEN HEADS OF JETTIES SHOWING SHOALING

C.27 The minimum navigation depth between the jetties is an important parameter to boaters. The monthly change in centerline controlling depth from 1972 to 1977 is illustrated in figure C30. The depth decreases at a linear rate from the 20-foot project depth after dredging each April to above MLLW on occasion by February. A slightly greater shoaling rate occurs between October and February. This figure indicates that the controlling depth does not suddenly decrease with a single storm, but steadily decreases after maintenance dredging. It also shows that the shoal develops relatively consistently through the years.

VOLUMETRIC ANALYSIS

C.28 The objectives of the volumetric analysis were to determine the quantities and rates of littoral transport and to determine the shoaling mechanisms. The history of the fillet impounded by the west jetty leads to a minimum estimate of the net eastward drift. Analysis of monthly volume changes documented by condition surveys in the navigation channel reveals the rate of shoaling as a function of time. The WES bypassing experiment and maintenance-dredging records are also presented in this section to develop a better understanding of how much material is in motion. Detailed description of the 1977-1978 winter is deferred to Section D where the results of the first year of the interim dredging contract are discussed.

C.29 West Jetty Fillet. A fillet accreted west of the west jetty after the jetty was constructed in 1962. Generally, the measurement of a fillet impounded by a long groin over a long time period is an excellent method of determining the net littoral-transport rate. This method was employed by planimetering areas of accretion on beach profiles taken between the San Lorenzo River and the west jetty. Volume changes were calculated by the average-end-area method. An October 1962 pre-project survey was compared with nine surveys listed in table C7. Profile area changes were measured between the berm at about +10 feet MLLW and the -20 foot contour. The 20-foot depth contour was selected based on review of profiles that indicated profile closure about this depth. Between the +10- and 20-foot

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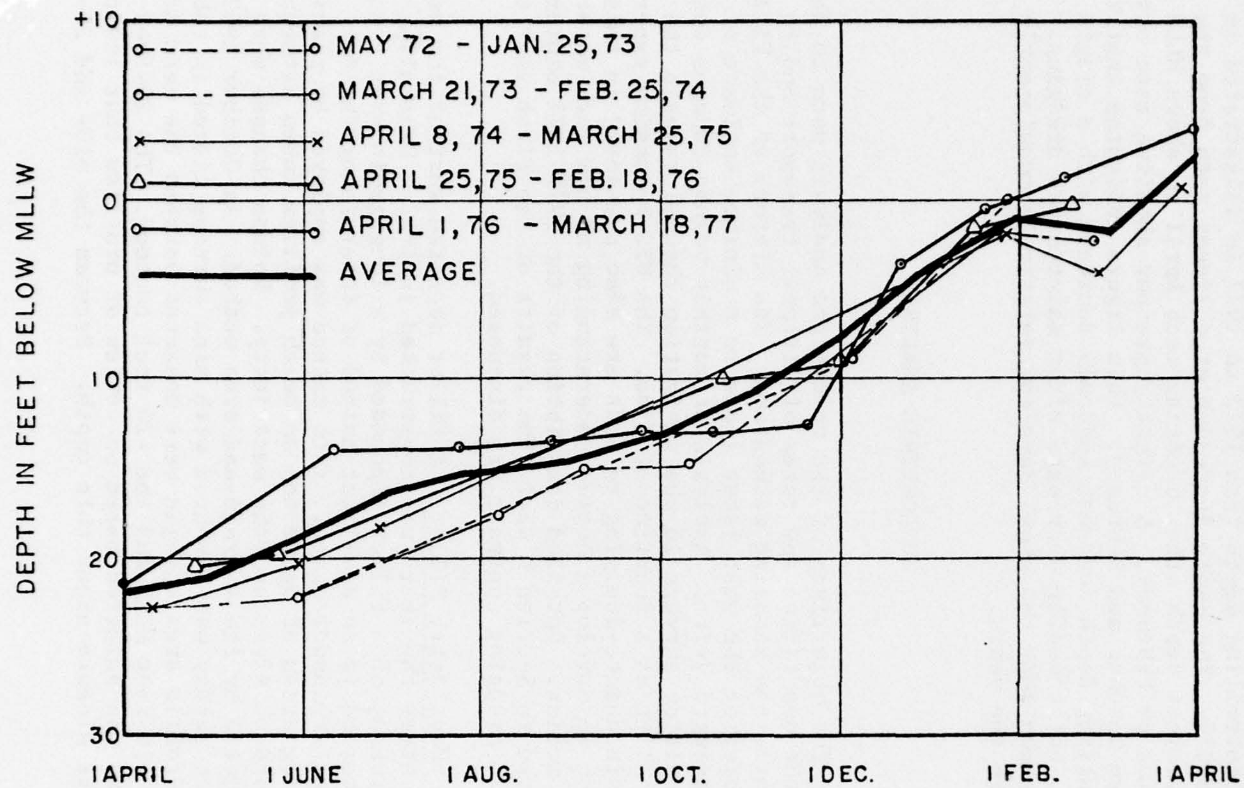


FIGURE C30. CHANGES IN CONTROLLING DEPTHS
(Centerline of Project Channel)
1972-1977

Table C7.

WEST JETTY IMPOUNDMENT VOLUMES

Date	Cumulative Volume (cu. yd.)	Differential Volume (cu. yd.)	Rate (cu.yd./yr.)
26 Oct. 1962	0		
		265,000	466,000
22 May 1963	265,000		
		- 55,000	-170,000
17 Sep. 1963	210,000		
		275,000	608,000
21 Feb. 1964	485,000		
		36,000	57,000
17 Oct. 1964	521,000		
		-129,000	-129,000
16 Oct. 1965	392,000		
		9,000	22,000
12 Mar. 1966	401,000		
		33,000	73,000
23 Aug. 1966	434,000		
		227,000	23,000
6 Aug. 1976	661,000		
		- 18,000	- 17,000
19 Aug. 1977	643,000		
1 Feb. 1978	652,000	9,000	2,000

depth contours, the bottom slopes are on the order of 1 on 30. The slope changes abruptly between the 18- to 20-foot depth contour and flattens to slopes gentler than 1 on 80. Also, in depths greater than 18 to 20 feet, the bottom fluctuates plus or minus two feet with no recognizable trends. These fluctuations could result from wave-induced motions of the survey boat. Changes in depths greater than 20 feet were not considered reliably accurate and therefore could not be used to draw significant conclusions.

C.30 The impoundment volumes measured between the San Lorenzo River and the west jetty are plotted in figure C31 as a function of time. Material was impounded at a rapid rate in the first two years. In the first seven months during the winter season a total of 265,000 cubic yards accreted. Through the summer 55,000 cubic yards eroded from the fillet, resulting in a net gain for the first year of 210,000 cubic yards. Between September 1963 and February 1964, spanning part of the winter, an additional 275,000 cubic yards accumulated. By October of 1964, 521,000 cubic yards had accreted over the two-year period, accounting for an average annual net impoundment of 260,000 cubic yards. The plot of impoundment volume versus time for the first two years during the winter months indicate short-term filling rates of 600,000 cubic yards per year followed by moderate losses each summer. The time plots of beach build-out in figure C25 show that the west fillet was nearly saturated by October 1964 and thereafter continued to accrete at a fraction of the initial filling rate. Maintenance dredging was performed in 1965 for the first time, indicating that sand had bypassed the west jetty during the first two years after the harbor was constructed. Despite the erosion of its entire beach face above MLLW during the 1978 winter, the fillet still continued to increase in total volume.

C.31 The close proximity of the San Lorenzo River, which has a highly variable flood and sediment discharge, complicates the analysis of the impoundment rate of the west jetty. The volumetric analysis indicated a rapid impoundment during the winters between 1963 and 1965 and erosion in the summers of 1963 and 1965. Detailed sedimentation studies of the San Lorenzo River for the period of record are not available; however, an estimate of the relative sediment discharge can be made by examining the

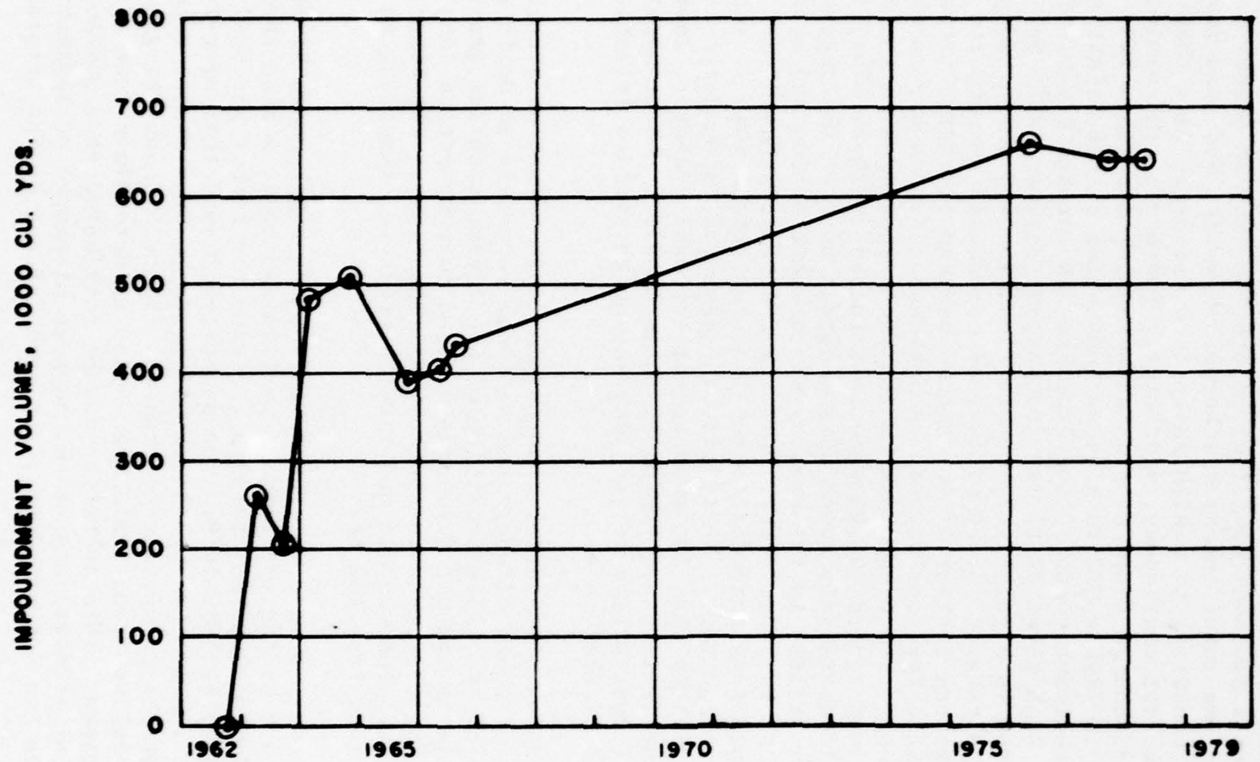


FIGURE C31. IMPOUNDMENT WEST OF WEST JETTY TO SAN LORENZO RIVER

stream-flow records. Two parameters that may be indicative of the River's sediment contribution are the maximum annual discharge rate and the total annual discharge. However, bed load movement is more sensitive to a high short-term discharge rate than it is to a low, but continuous, discharge. Therefore, the maximum annual discharge rates of the San Lorenzo River since 1937 are plotted in figure C32 to give an indication of the relative annual sediment contribution from the San Lorenzo River. The initial high rates of fillet impoundment may partially be attributed to a relatively severe flood in 1963 which had a peak discharge rate of 13,400 cubic feet per second as compared to the long-term average of the annual maximum discharge rates of 7,961 cubic feet per second. A reasonable hypothesis is that a delta was formed by sediment which had accumulated in the riverbed since flood control works were constructed in 1958. The fine material was rapidly moved offshore, but much of the coarser material drifted to the east by winter-wave action and became trapped by the west jetty. The river delta continued to contribute more material to the littoral stream than it contributes in an average year. Therefore, the measured impoundment rates may be greater than the average net littoral transport rate. Also, the jetties are too short to impound all the transported material over a time period long enough to yield a reliable estimate of that rate.

C.32 Losses of material reported in table C7 may be attributed to loss of fines offshore and seasonal reversals in drift rates. Because of the close proximity of the river, material impounded by the jetty may not have been exposed to wave action long enough for the waves to sort out all the fines. The fines were eventually winnowed out and carried offshore, resulting in a volume loss from the beach.

C.33 Profiles were not taken west of the San Lorenzo River. The growth of the westernmost profile indicates a long-term accretion of 300 feet between the time the harbor was built and the winter of 1977-1978. The quantity of sand that accreted west of the San Lorenzo River, along Santa Cruz Beach, cannot therefore be accurately determined. Plate 5 shows the growth of the shoreline along Santa Cruz Beach as determined from aerial photographs. This accretion can be attributed to a number of different littoral processes, such as discharge of sediments from the San Lorenzo River, the groin effect of the natural rock

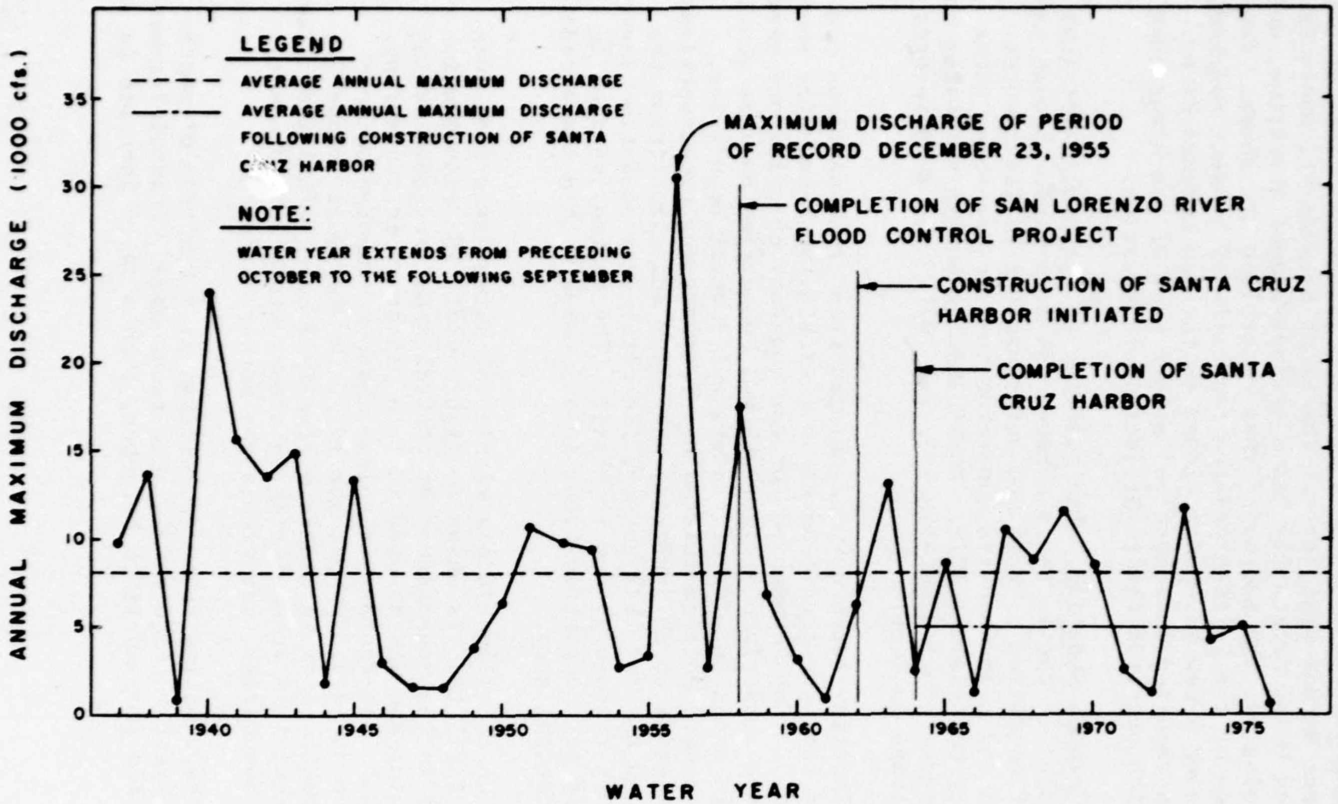


FIGURE C32. ANNUAL MAXIMUM DISCHARGE OF THE SAN LORENZO RIVER

prominence just east of the River, and the groin effect of the Santa Cruz Harbor west jetty. The beach between Pt. Santa Cruz and Black Point originally had a cresent-shaped shoreline, and Black Point was the end-point that controlled its shape. Construction of the jetties created two cells of cresent beaches, interposing a new end-point closer to the San Lorenzo River. This has resulted in accretion farther west of the River than that which occurred before project implementation.

C.34 Channel Shoaling. The quantity of littoral drift that has shoaled in the entrance channel was determined through analysis of condition surveys and maintenance-dredging records. Detailed and comprehensive condition surveys have been taken on a monthly basis since 1972. The following analysis focuses on the five-year period between 1972 and 1977. The 1978 records are analyzed in Section D.

C.35 Shoaling rates were calculated from cross-sections constructed from the condition surveys within the boundaries shown in plate 9. The boundaries for the quantity calculations extend beyond the authorized project channel boundaries because shoaling occurs in areas outside of the project channel boundaries. Volume changes were calculated by planimetering cross-sectional areas relative to an arbitrary base line and normalizing the results to the May 1973 survey, the first year condition surveys were taken frequently and in detail. The successive profile lines did not completely close in all cases within the boundaries of the control area.

C.36 The shoaling history within the survey area from June 1972 to April 1978 is shown in figure C33. The accumulation of material in the measurement or control area increased during each winter and was dredged in late winter or in the spring. The dashed line represents volume changes computed between condition surveys. By October 1977, the residual volume was 50,000 cubic yards in excess of the 1973 base. This indicates that material is accumulating in areas outside the project channel boundaries, but within the harbor entrance.

C.37 The shoaling rates are plotted as a function of months for each year in figure C34. The rates show a gradual increase in shoaling from April to December, then a rapid increase in

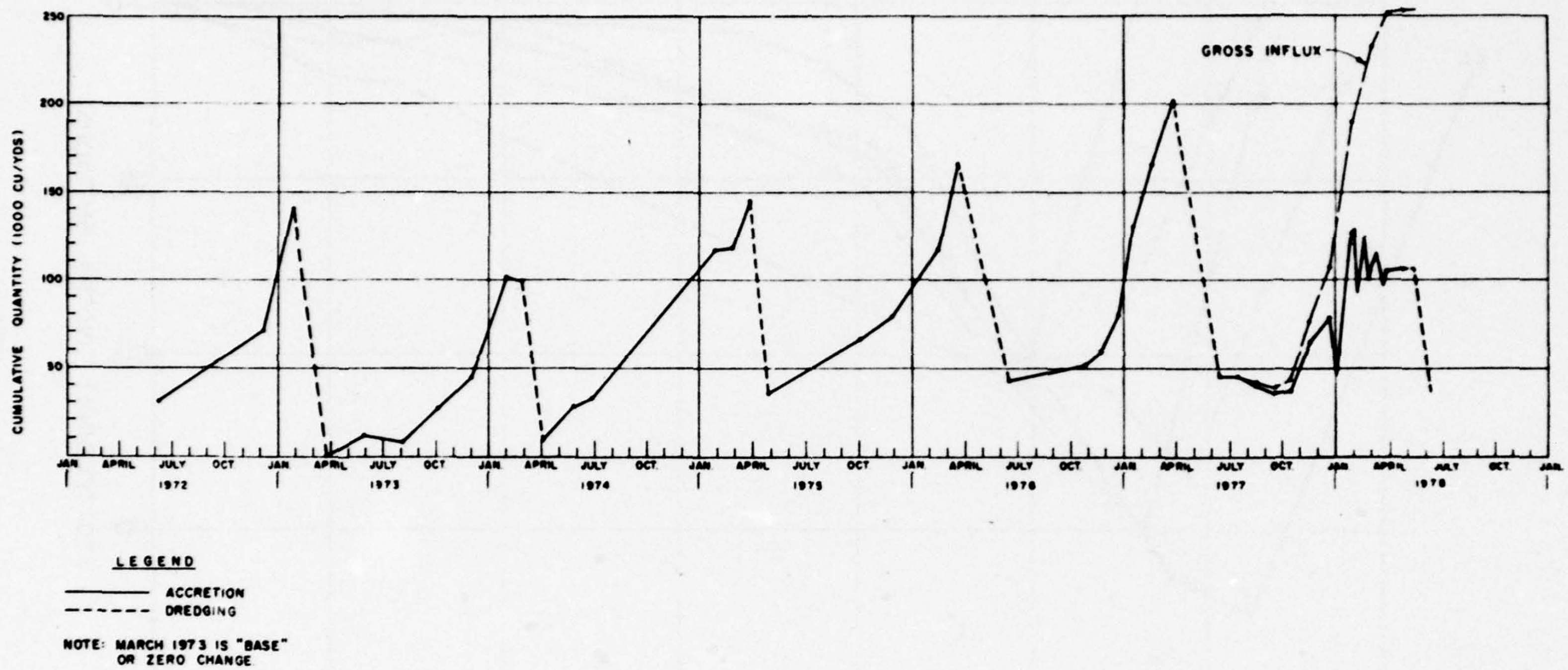


FIGURE C-33 SHOALING HISTORY

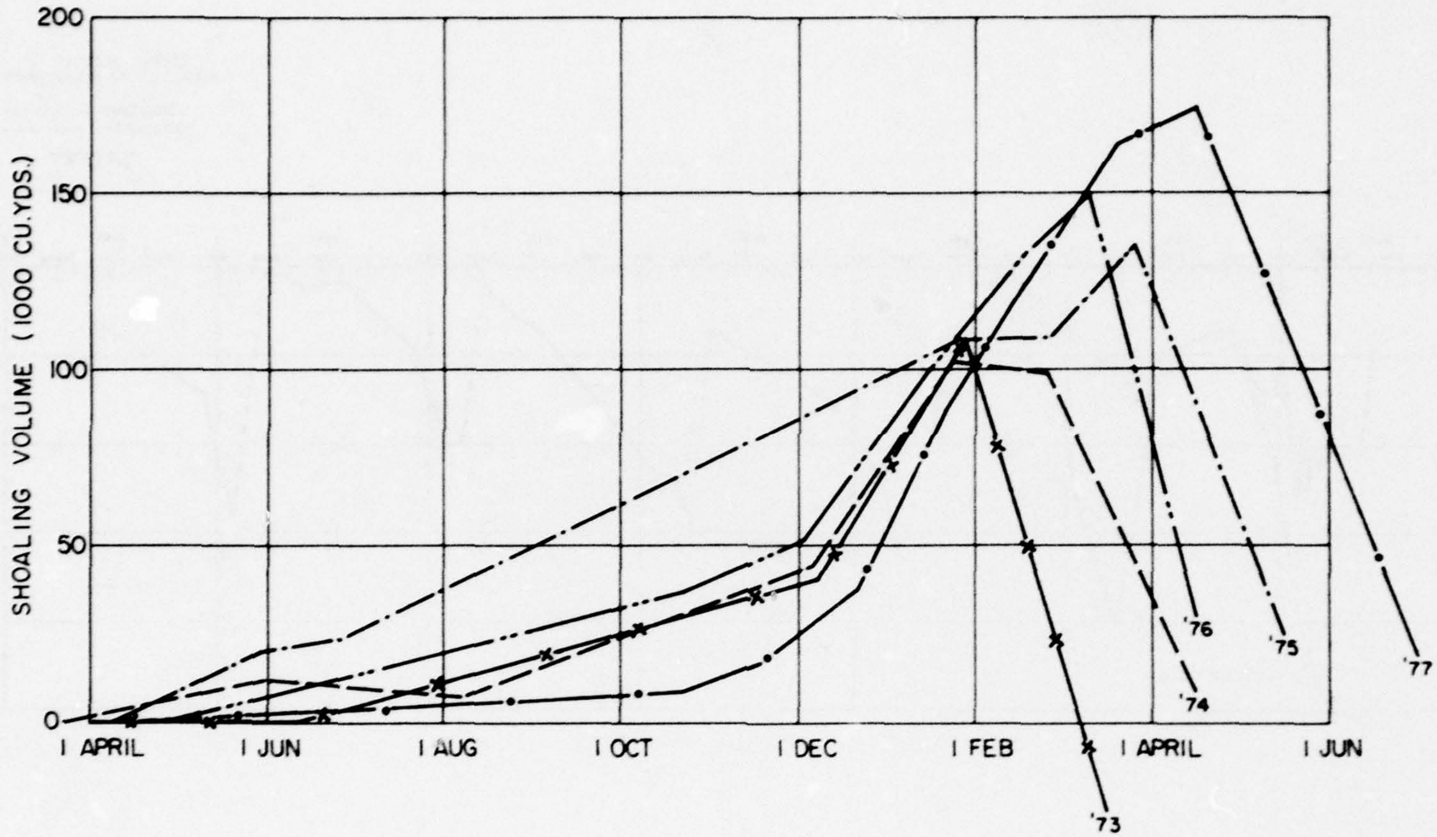


Figure C34. COMPARISON OF ACCRETION AND DREDGING QUANTITIES BY MONTH (1972-1977)

Note: Day zero is completion of previous dredging episode.

shoaling from December until the maintenance dredging episode. The 1975 line shown is a straight-line interpolation between July and February, but the actual shoaling was presumed to follow trends similar to those of other years. The plot also shows that the period of maintenance dredging has varied. Dredging was initiated in February 1973 and completed before April 1973. The most active shoaling occurs during these months. When dredging was performed during this time period, considerable quantities of material may have entered the area during the dredging episode. The 1977 dredging episode was the latest annual dredging episode of record, and it may reflect the sand-trapping potential of the harbor more than the dredging eipsodes of other years because the harbor was allowed to shoal for a longer period before dredging was initiated. Figure C35, which plots volume shoaled as a function of the number of days after dredging, exhibits a greater scatter of plotting points than figure C34. During the 1976-1977 winter season, the WES experimental bypass plant removed an estimated 45,000 cubic yards from within the control areas.

C.38 Shoaling rates shown in figure C34 occurred in consistent seasonal patterns with little variation from year to year. Figure C36 presents a smoothed shoaling-rate curve giving the average monthly accretion rates and the upper and lower bounds for the five-year period from 1972 to 1977. Monthly shoaling rates derived from figure C36 are summarized in table C8.

C.39 Dredging History. Two types of dredging operations have been in effect at Santa Cruz: channel maintenance and experimental bypassing. Annual maintenance dredging was initiated in 1965 to remove sand from the project channel. The experimental bypassing operation was a field test of a fixed-plant jet-pump (eductor) system conducted by Waterways Experiment Station (WES). This operation was initiated in the winter of 1975-1976 to remove sand from areas inside the entrance but outside the Federal-project channel limits, and from the updrift beach. The experiment was completed in March 1978.

C.40 The maintenance dredging program comprised mobilizing a hydraulic dredge in the late winter to clear the channel by the end of spring. Typically, a 12-inch hydraulic pipeline dredge was used to discharge material 1000 feet east of the east jetty. After each demobilization, the channel slowly shoaled

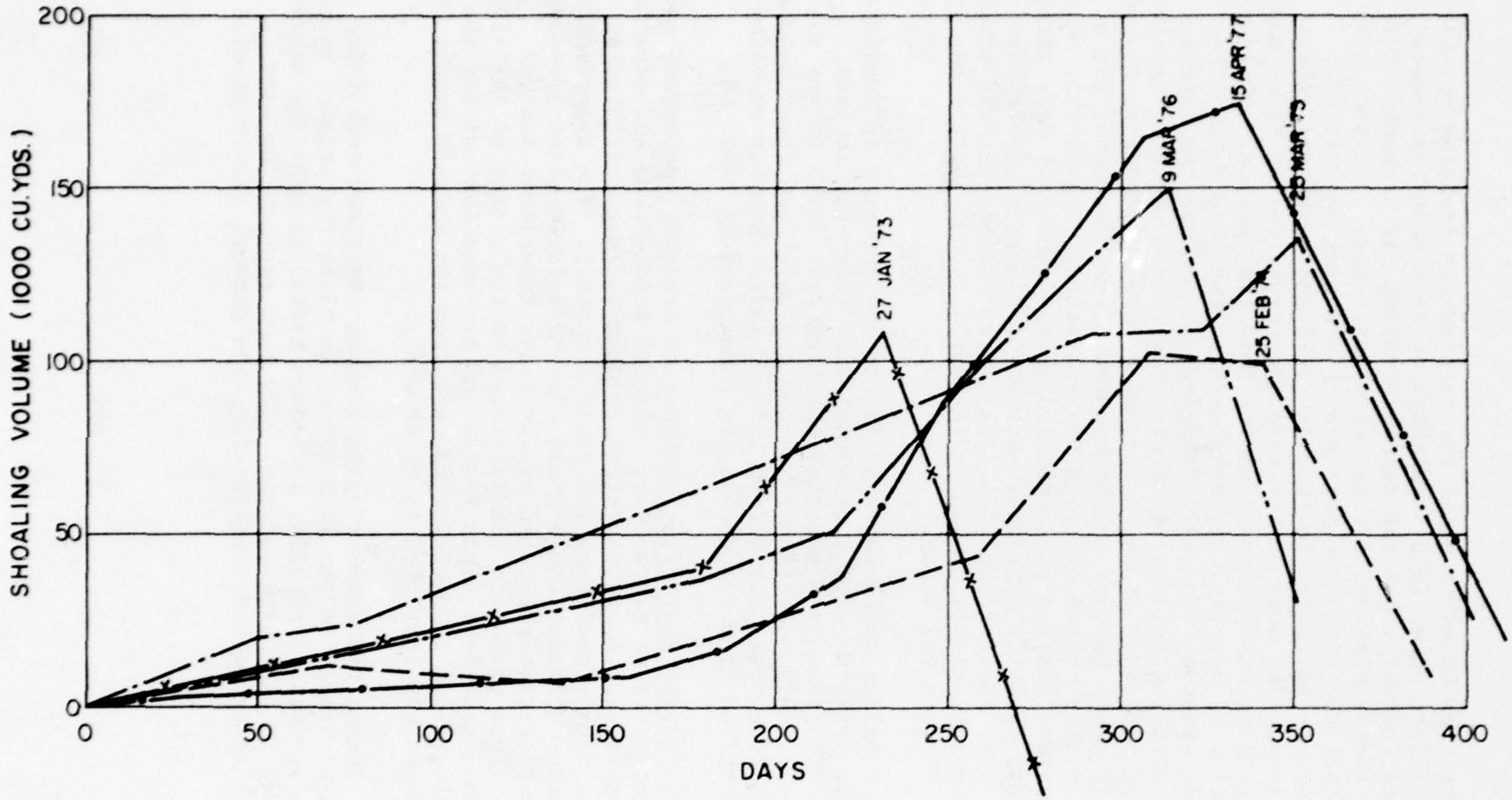


Figure C35. COMPARISON OF ACCRETION-DREDGING QUANTITIES (1972-1977)

Note: Zero quantity and day is from completion of previous dredging episode for each year.

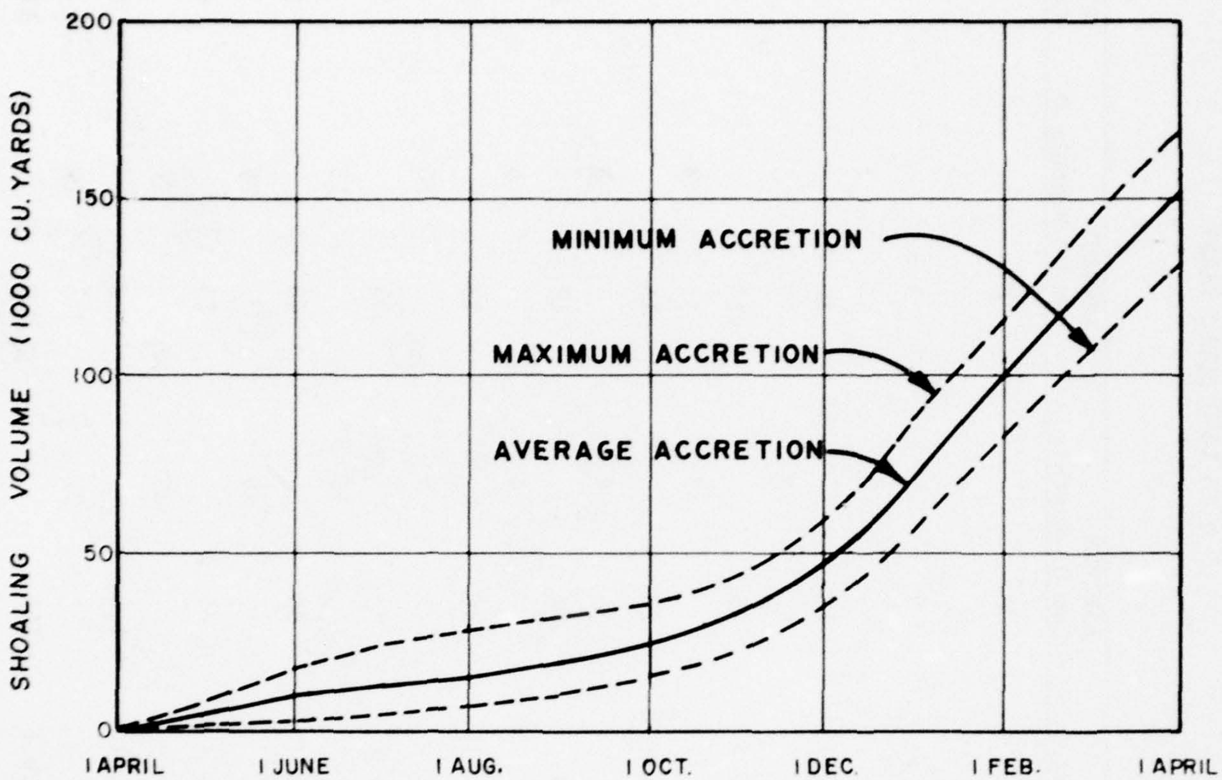


Figure C36. AVERAGE, MAXIMUM, AND MINIMUM MONTHLY CUMULATIVE RATES OF ACCRETION BETWEEN DREDGING EIPSODES 1972 to 1977.

Table C8

MEAN MONTHLY SHOALING RATES
BASED UPON 5-YEAR PERIOD 1972-1977
Q IN 1000 CUBIC YARDS

Date	MAXIMUM Cum Q	MAXIMUM Monthly Q	AVERAGE Cum Q	AVERAGE Monthly Q	MINIMUM Cum Q	MINIMUM Monthly Q
April 1	0		0		0	
		9		5		
May 1	9		5		2	
		9		5		1
June 1	18		10		3	
		7		3		2
July 1	25		13		5	
		3		3		2
August 1	28		16		7	
		4		3		3
Sept. 1	32		19		10	
		4		6		5
Oct. 1	36		25		15	
		9		10		8
Nov. 1	45		35		23	
		15		13		12
Dec. 1	60		48		35	
		29		25		33
Jan. 1	89		73		68	
		29		27		17
Feb. 1	118		100		85	
		27		29		25
March 1	145		129		110	
		25		28		24
April 1	170		157		134	

during the summer, building a bar across the entrance channel. The rate of shoaling increased rapidly in November, and only a small tidal channel remained open from January until the maintenance dredging was resumed. This dredging program was modified in 1977 when a series of multi-year, phased dredging programs were implemented. Under the first two-year contract, material was dredged in phases during the 1977-1978 winter to maintain the channel open to navigation throughout the year. This will be repeated in the 1978-1979 winter.

C.41 Maintenance-dredging records are summarized in table C9 and plotted in figure C37. Pay yardage represents the quantity of material that the Government agreed to pay the contractor, which was not necessarily equal to the quantity of material removed. The dredgerman estimated that they dredged as much as twice the pay yardage prior to 1979 when pay quantities were determined by comparing pre-dredging and post-dredging surveys. This gave erroneous results when the dredging lasted several months in late winter and spring when shoaling occurred during the dredging operation. Past practice required the contractor to bid on shoal quantities determined from a November survey. Pre-dredging surveys taken in January or February were then used as a basis for determining quantities of pay yards. Considerable shoaling occurred prior to and during the actual dredging operation and in some cases after dredging, prior to the post-dredging survey. The longer the dredging operation and the smaller the pumping capacity of the dredge, the greater was the potential for dredging more yardage than the Government allowed for payment. In the spring of 1977, the dredging logs indicated that a 12-inch dredge removed 300,000 cubic yards, but the contractor was paid for removing only 148,000 cubic yards. The bid was based on 62,000 cubic yards.

C.42 Review of figure C37 shows that the initial quantity of dredged material in 1965 was large. The quantity decreased in 1966 and increased thereafter until 1970 after which the pay yardage averaged about 100,000 cubic yards, with little fluctuation. The relatively large amount dredged in 1965 includes shoaling that occurred during two post-construction years. The increase in pay yardage with time up to 1970 reflects the decreasing capacity of the west fillet to store material.

Table C9. MAINTENANCE-DREDGING "PAY YARDAGE"

Date	Cubic Yards Dredged
May-August, 1965	70,000
June-July, 1966	34,000
May-June, 1967	57,000
April-May, 1968	60,500
March-April, 1969	79,000
April-May, 1970	94,700
May-June, 1970 (Clamshell dredging to remove kelp mat)	11,500
May-June, 1971	108,300
May-June, 1972	90,000
Feb-March, 1973	109,000
March-April, 1974	60,000
April-May, 1975	91,000
March-April, 1976	98,000
April-June, 1977	147,000
Dec.-June, 1978	162,000

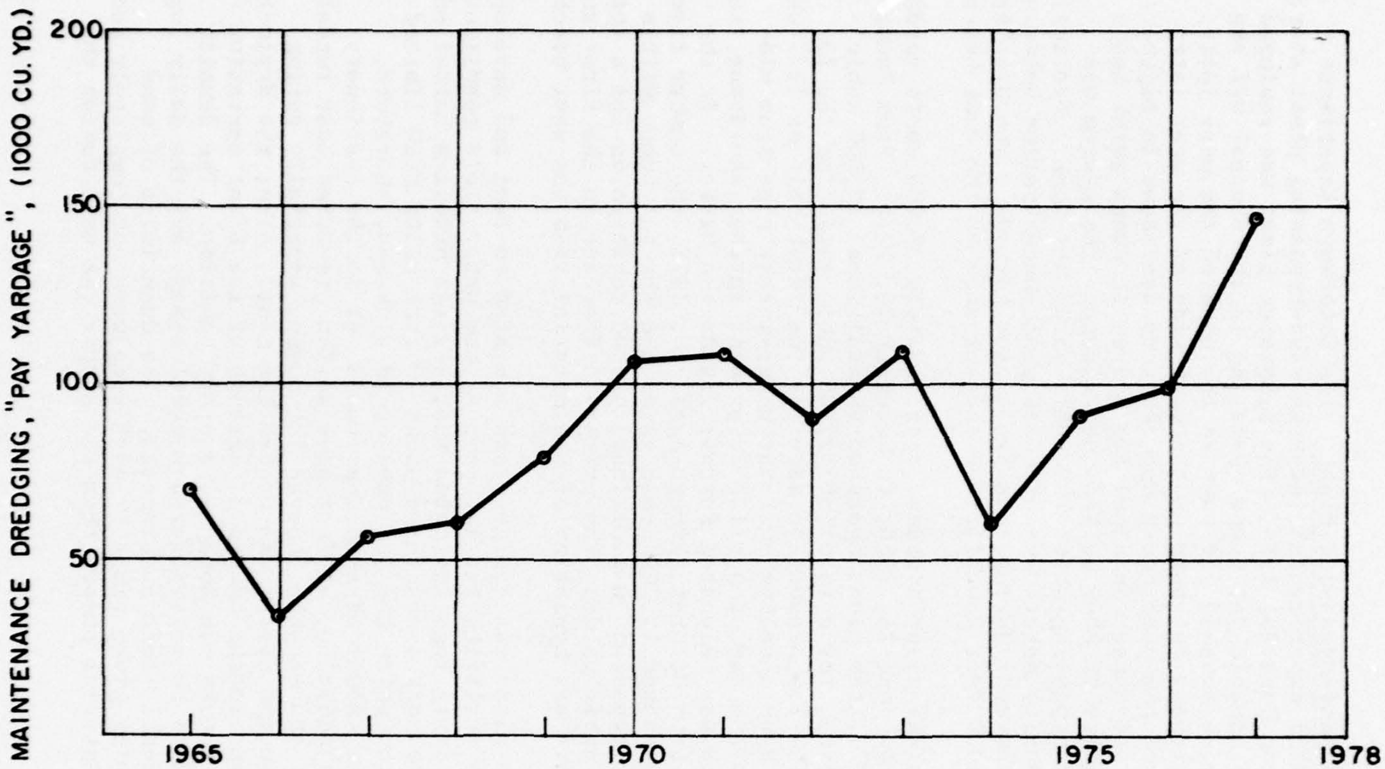


FIGURE C37. ANNUAL MAINTENANCE DREDGING HISTORY

C.43 <u>WES Sand-Bypassing Plant</u>. The Waterways Experiment Station (WES) experimental jet-pump sand-bypassing plant started operating on June 26, 1976. The bypassing plant was equipped with three movable jet pumps operating in the channel off the west jetty, a movable jet pump at the head of the west jetty, and a stationary jet pump on the west side of the west jetty. Locations of the pump house and eductors are shown in figure C38. The pumphouse contained two hydraulic pumps which had a total capacity of 2000 gallons per minute. The system was capable of bypassing about 100 cubic yards per hour. Vertical nuclear density meters measured sediment concentration between 12 and 15 percent during peak production periods. The discharge area was 1000 feet downdrift of the entrance on the east beach.

C.44 The WES plant bypassed approximately 37,000 cubic yards of sand from June 26, 1976, to December 20, 1976. From January to July 1977, the plant bypassed an additional 12,000 cubic yards of sand, for a total of 52,000 cubic yards for the 13 months. The low production rate for the first half of 1977 was caused by sand shoaling over the water-intake pipe from mid-January to the end of April. To prevent further shutdowns, the intake pipe was relocated farther inside the harbor. In the next year from October through April 10, 1978, the number five jet pump bypassed 12,500 cubic yards and the jet pumps within the harbor bypassed an additional 42,500 cubic yards for a total of 55,000 cubic yards. The primary difference in the first and second years was bypassing of the material from the west beach.

C.45 The plant was designed and operated to test and develop techniques utilizing the jet-pump system under field conditions. Review of daily logs indicates that several problems curtailed output. Bedrock encountered about 20 feet below MLLW limited the depth to which the jet pump could sink and, therefore, reduced the amount of sand that could get to the stationary pumps. In addition, violent wave action prevented easy repositioning of the movable, buoyed jet pumps, especially during storms when the system was needed the most. Also, the drylocking of the water intake caused a shutdown of the plant operation until the intake was moved to another position. The density meters had to be recalibrated several times, but the daily logs were apparently left uncorrected. The quantities of sand pumped during given times by each pump were not completely documented. Material pumped from the number one unit during the

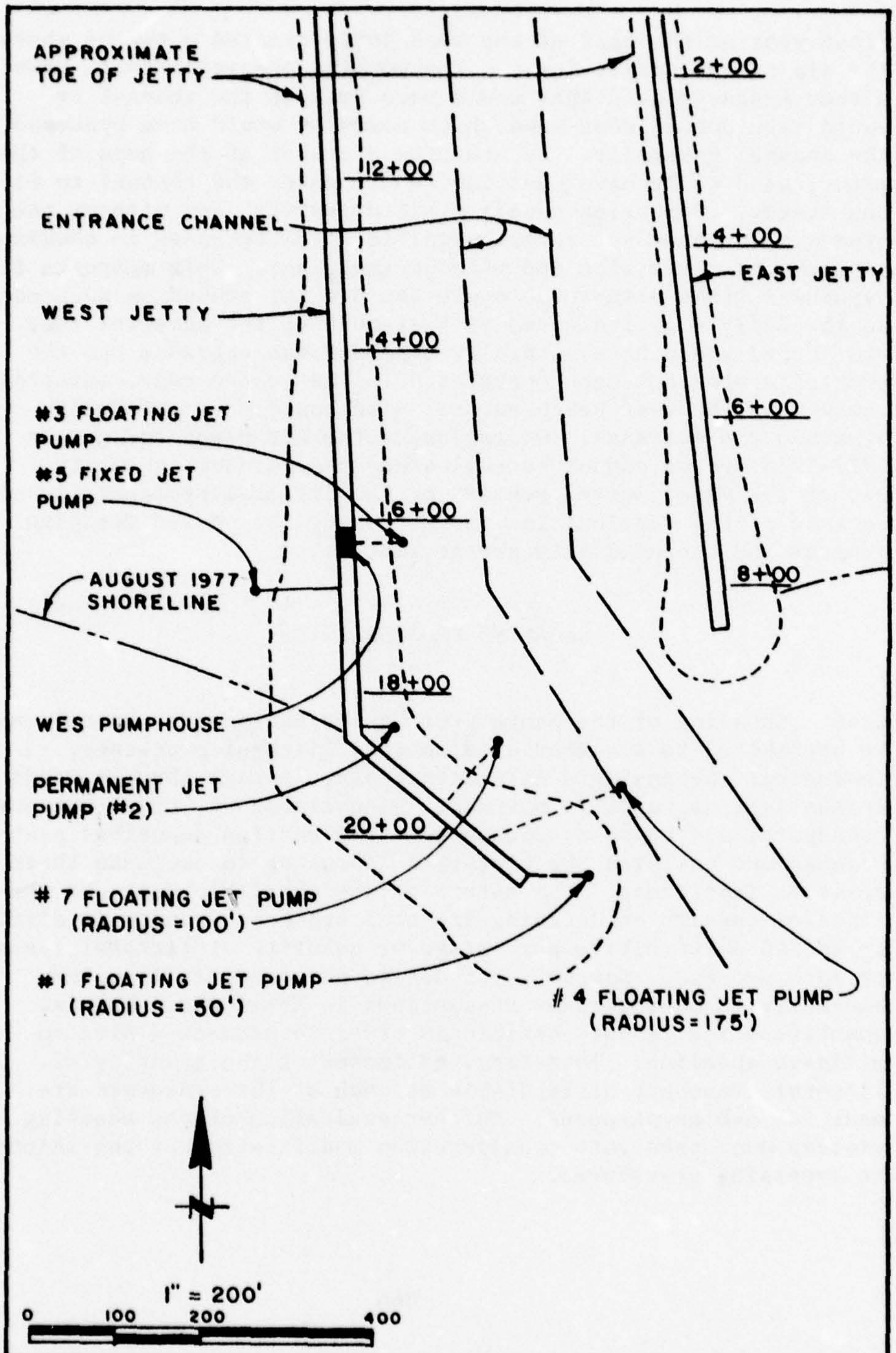


FIGURE C38. WATERWAYS EXPERIMENT STATION (WES) JET-PUMP SAND-BYPASSING SYSTEM

first year at the head of the west jetty created a crater where the tip shoal usually forms. The pumping operation could have either bypassed sand that would have entered the channel or could have pumped some sand which normally would have bypassed the channel naturally. By creating a crater at the head of the jetty, sand could have been induced to enter the channel to fill the crater. Comparing shoaling histories with and without the bypass plant does not reveal significant differences in shoaling patterns or rates with and without the plant. This supports the hypothesis that either the operation did not pumped as much sand as the daily logs indicated or that much of the material that was pumped would have naturally bypassed the entrance had the pumping system not been operated. In the second year, material removed by the west beach eductor also could have naturally bypassed the entrance. Operation of the WES plant during the 1977-1978 season cannot be evaluated to accurately quantify either the experimental results or the littoral processes involved because of the complexities introduced by the phased dredging program and the unusually severe winter.

SHOALING PROCESSES

C.46 Shoaling of the Santa Cruz Harbor navigation channel can be attributed to a number of disparate littoral processes, including: bypassing a saturated groin, leakage through voids in the jetties, updrift movement, wind transport, tidal-current transport, and onshore transport. This section describes each process and analyzes the history of shoaling to estimate their relative importance. The nature of the available data and the state-of-the-art of defining littoral processes render it difficult to assign a definitive percentage or quantity of littoral transport to each process. However, for design purposes, it is often necessary to make certain assumptions in areas where precise quantification is not possible in order to prepare a plan to mitigate shoaling. Therefore, estimates of the quantity of littoral transport attributable to each of the processes are made for design purposes. Further evaluation of the shoaling process must take into consideration modification of the maintenance or bypassing procedures.

C.47 Bypassing a Saturated Groin. Sand bypassing the head of the west jetty is analogous to the classical case of sand bypassing a saturated groin. This process is believed to be the primary source of channel shoaling. Immediately after construction of the west jetty, a fillet formed on its west side at the rate of 250,000 cubic yards per year for two years, and beaches east of the harbor entrance experienced severe erosion. The shoaling occurred at a rate of about 600,000 cubic yards per year during the winter. Channel maintenance dredging was at a low rate during the first two years, averaging 35,000 cubic yards per year.

C.48 The minimum net littoral-transport rate at the harbor site is estimated at 285,000 cubic yards per year, based on the early growth rate of the fillet and assuming all maintenance dredging was from bypassing the west jetty or transmission through and over it. Within three years after construction of the jetties, the west fillet had grown to the angle point of the doglegged west jetty. As the west beach continued to move seaward during this period, maintenance dredging pay yardage increased to quantities more than 100,000 cubic yards per year, fillet formation decreased to less than 25,000 cubic yards per year, and erosion of the downdrift beaches decelerated when they were nourished with the larger amounts of dredge spoil that became available in later years. The water depth at the head of the west jetty decreased rapidly, and within a few years sand bypassed the head at a greater rate than it was being trapped in the fillet. The method by which sand entered the channel can be gleaned from a review of the monthly condition surveys taken since 1972. Plates 7 and 8 summarize two years of this analysis. A tip shoal extended progressively from the head of the west jetty across the channel to the head of the east jetty. A small tidal channel remained open along the east jetty. The cross-sectional area of this channel compared with the area predicted by the O'Brien formula (1968). Evidence of littoral drift entering the channel from the east or from tidal-induced currents cannot be found by observing the growth of a shoal in the channel or of sand build-up along the east jetty. The formation of the shoal across the navigation channel permitted littoral drift to naturally bypass the channel. The shoal had been present for most of the winter months when storms actually had the greatest potential for littoral transport. This suggests that considerable

bypassing must have occurred and that the dredging quantities represent low estimates of the gross littoral transport.

C.49 The theoretical net longshore transport rate determined by energy-flux calculations is 488,000 cubic yards per year toward the east. The gross transport rate is estimated to be 1,168,000 cubic yards per year. The total longshore-transport rate is related to energy flux by an empirical factor; however, the distribution of littoral transport through the surf zone is not well known. Longuet-Higgins (1970) formulated the longshore current-velocity distribution through the surf zone. Komar and Inman (1970) related sediment transport to the longshore current-velocity distribution, and Komar (1971) summarizes the state-of-the-art for estimating the longshore sediment-transport distribution. The total load comprises suspended and bedload. The suspended load is about 20 percent of the total and has been found to occur primarily within the surf zone.

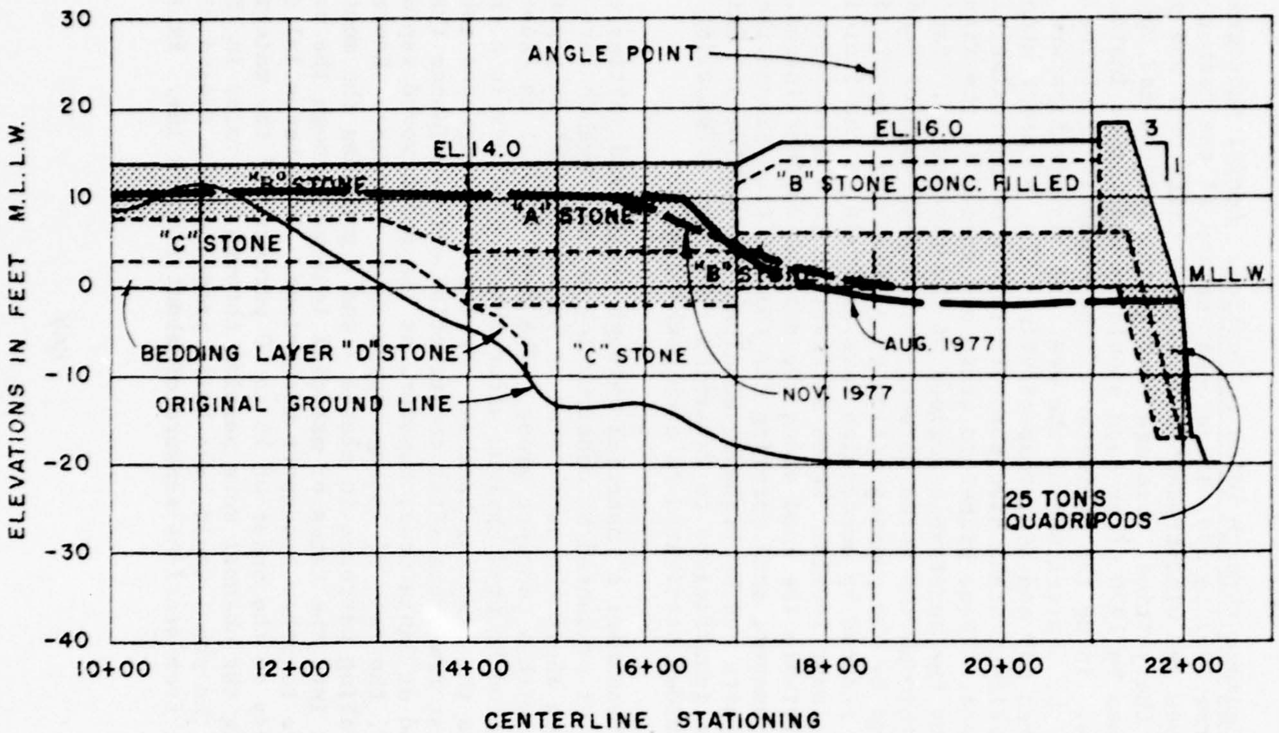
C.50 Approximately 80 percent of the sediment transport occurs within the surf zone, and 95 percent occurs within one and one-half surf-zone widths. At Santa Cruz, waves break with heights of less than 20 feet 98.9 percent of the time, and 80 percent of the sediment transport occurs on the steep nearshore slope at depths of less than 20 feet. The predominant littoral forces, however, are not produced by the exceptionally high waves of short duration, but by smaller waves of greater duration. Wave heights of 8 feet are exceeded 17 percent of the year. Under these conditions, less than 1 percent of the littoral drift is transported in depths greater than 20 feet. Wave gage data indicate that these statistics may greatly over-estimate the occurrence of high-wave activity.

C.51 The preceding theoretical considerations indicate that the primary zone of littoral transport is on the steep 1 on 30 nearshore slope between the berm and the 20-foot depth contour. When constructed, the head of the west jetty was in 20 feet of water, and initially the jetty intercepted littoral drift quite efficiently. However, figure C27 indicates that by 1969, the shoal off the outer leg of the jetty extended from the -2-foot elevation at the top of the core of the jetty to the 20-foot depth contour. Waves breaking on this slope could readily transport large volumes of sand both through voids in the outer leg of the jetty and around its head. Sand bypassing the

head of the jetty can both enter the channel and bypass the entrance and deposit on the downdrift beach. The tendency to bypass the entrance increases as the shoal in the channel builds.

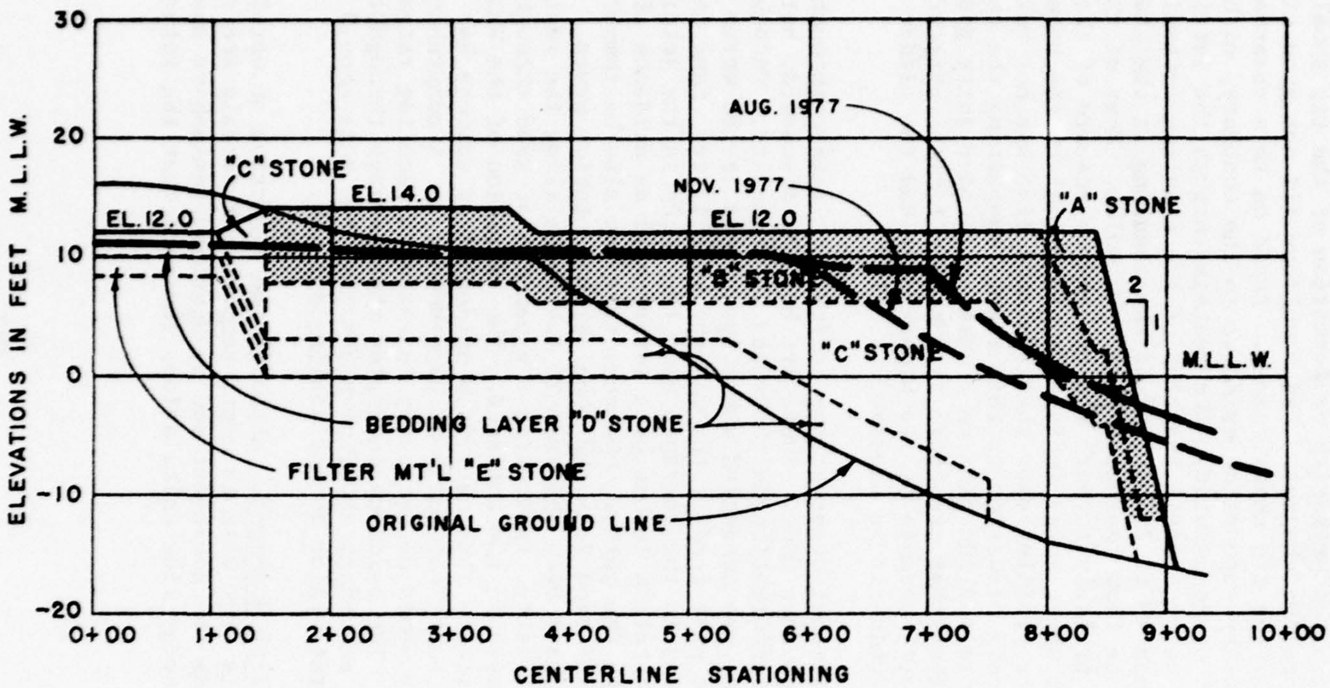
C.52 Leakage through Voids in Jetties. Aerial photographs taken from 1963 to 1967 after construction of the harbor show that waves and wind transported littoral drift over and perhaps through the jetties. Leakage through the jetties had apparently diminished by 1970 after sand shoulders had formed inside the entrance, lining the perimeter of the channel. Profiles taken through the centerlines of the west and east jetties are shown in figures C39 and C40, respectively. The "A" armor stone and "B" underlayer stone have porosity ratios of approximately 35 to 40 percent. These high-void areas are shaded in the figures to highlight the penetrable regions of the structures. Sand can be washed through the voids directly by flow-through of sand-laden water and by the pumping action of alternating pressure differentials induced by successive wave arrivals on the fluidized sand at lower levels. Waves diffracting in the entrance channel can distribute the sand along the channel farther inland. In a similar manner, sand entering the channel from around the head of the jetty can be transported along the shoulders. Existing data are insufficient to determine the rates of shoaling along the shoulder attributed to each mechanism.

C.53 Transport of material through groins and jetties can be reduced or prevented by constructing an impermeable core to an elevation above the swash zone or by sealing existing porous jetties with a concrete grout. The need to seal an existing jetty often becomes obvious after shoaling occurs in a channel opposite the flanking beaches. Review of photographs taken in the first few years after construction offer evidence that this occurred at Santa Cruz; however, as the beach moved rapidly seaward, the rate of leakage appeared to decrease. Review of the shoaling patterns in plates 7 and 8 provides the most reliable insight into the rates of material leakage through the voids relative to other transport mechanisms. If substantial quantities, say on the order of 25 to 50 percent, of the material shoaling the channel were passing through the voids in the west jetty, the shoal would have built progressively toward the channel from stations seaward of about station 16+. Examination



℄ PROFILE WEST JETTY

FIGURE C39



℄ PROFILE EAST JETTY

FIGURE C40

of the shoaling contours in plates 7 and 8 and review of the shoaling patterns in other years indicate that instead, the shoaling occurred primarily by formation of the tip shoal which migrates into the channel. The shoulders did not grow significantly until the tip shoal formed. Based on this observation, and lack of any supporting evidence to the contrary, either theoretical or documentary, the leakage through the jetties is considered to be a small fraction of the shoaling mechanism. Approximately 10 to 20 percent of the shoaling in the channel is assumed to be from leakage through the voids. Most of the leakage is probably through the breakwater seaward of the angle point where waves impinge nearly normal to the breakwater. The offshore profiles show that the sand line has not built above the core elevation in this reach, supporting the concept that the core may function as a weir. The east jetty profile indicates that that structure is a more efficient barrier than the west jetty because of its higher core and the milder wave climate to which it is exposed.

C.54 Additional observations indicating leakage through the jetties have been made. These are briefly discussed, but they provide no information on the rates of transport. Undocumented dye studies were conducted, where dye placed in the water was observed to pass readily through the west jetty. Sand in suspension observed in the entrance near the voids in the jetties seaward of Station 16 has been interpreted as evidence of transport through the jetty. However, this may also be temporary suspension of sand from the shoulder by interior waves. Small sink holes have been observed in the beach along the west side of the west jetty, indicating the pumping of sand through the voids by waves in the entrance. The operation of the WES eductor system indicates that sand from unidentified sources has refilled the crater near Station 16 at pump number 3. A comprehensive field study would be necessary to estimate shoaling rates through the voids. The evidence indicates that leakage through the voids is a secondary shoaling mechanism, but this should be checked in more detail prior to any sealing attempt.

C.55 Updrift Movement. Although the direction of net littoral transport is from west to east, temporary reversals are caused by seas from the southeast and by Southern Hemisphere swell. The wave energy-flux calculations indicate that the ratio of

westward to eastward longshore transport potential is 0.4. Over one-half of the westward energy flux occurs during January and February as a result of locally generated southeasterly seas. This indicates that there is a large possibility for sand to enter the channel during reversals. However, the quantity of material entering the channel by this process is estimated to be considerably less than 40 percent of the total. During the winter when the shoaling occurs, the beach normally recedes along the east jetty. This recession is a realignment of the littoral cell in response to the northern Pacific swell. Any sand that is moved in the updrift direction is usually trapped by the jetty during short periods of storm activity. Movement of suspended sand seaward of the east jetty head, by rip currents and by saltation-transport across offshore bar or shoal formations, may account for the passing of some material into the harbor. The depths off the head of the east jetty normally increase in the winter, as shown in plates 7 and 8. This indicates predominance of eastward transport in the winter and therefore less potential for updrift transport to shoal the entrance than the energy-flux analysis indicates.

C.56 During February 1978 high seas from the southeast made it necessary to close the harbor to navigation. A large eddy of discolored water was observed on the east beach, and it was feared that the channel would shoal in direct response to this. However, informal sounding by the harbor master detected no bar off the east beach nor significant shoaling in the harbor channel. This supports the theory that updrift movement is not a major direct cause of shoaling. A few days after the southeasterly storm, a northern hemisphere swell did shoal the channel but this was due to downdrift, not updrift movement. Nonetheless, when dredged material is deposited on the east shore during the winter, the east beach builds seaward, and this does increase the potential for sand to be carried westward back into the harbor. Therefore, the phase relationship between dredging and storm activity will influence the rate that material returns to the harbor.

C.57 During the summer, the east beach near the east jetty builds seaward in its normal realignment due to the change in wave climate. The water depth at the head of the jetty shoals to 5 feet or less. This gives the appearance that reverse bypassing is greatest during a southeasterly sea or a southern

hemisphere swell in the summer. Shoaling records, however, indicate that only minor rates of shoaling from the east occur during the summer. The tip shoal formation off the west jetty and its growth during the winter support the contention that most of the shoaling occurs as a result of material bypassing the head of the west jetty. Based on the above discussions, the amount of littoral drift bypassing the head of the east jetty in reverse longshore transport is estimated to be on the order of magnitude of 10 to 15 percent of the total drift entering the harbor. This rate is variable and is probably greater when a late spring dredging episode discharges material close to the east jetty.

C.58 Wind Transport. Aerial photographs taken in 1963 show evidence of wind transport over the west jetty. A parapet constructed on the west jetty has trapped wind-driven sand. The wind-driven sand has also built low-lying dunes on the upcoast beach. The slow formation of shoals in the channel where the sand is being transported by the wind indicates that the rate of wind transport over both the east and west jetties is minor. The potential for wind-blown transport was calculated by using the method proposed by Johnson and Kadib (1965). A wind rose obtained from the Natural Bridges Park LEO station from April 3, 1968, to November 24, 1970, was used to calculate the transport over the west jetty and east jetty. The calculations indicate a potential for 6600 cubic yards per year to be transported eastward over the west jetty and 2200 cubic yards per year to be transported westward over the east jetty. The formation of the small dunes on the west side, the presence of considerable debris on the beach occasionally, the trapping effect of the small parapet on the west jetty, and the lesser degree of wind exposure of the harbor relative to that of Natural Bridges Park, supports lowering the calculated estimates to 5000 and 2000 cubic yards per year, respectively.

C.59 Onshore-Offshore Transport. Beach and offshore-bottom profiles are dynamic and seldom reach a true state of equilibrium. Changes in wave climate induce changes in the alignment of the beach as well as its profile. High, steep waves generally strip material from the beach face and transport it offshore, often forming bars and shallow troughs. The bars induce waves to break offshore and reduce the amount of wave energy transmitted to the beach. This process continues until the beach reaches a new quasi-equilibrium. During episodes of low-steepness waves, the bar and offshore materials are transported back onshore.

This mechanism probably occurs at the project site. Review of profiles taken to depths of 40 feet has not revealed the presence of a bar. The nearshore bottom slope to the 20-foot depth appears to be too steep for a bar to form. The slope is very flat in depths greater than 20 feet, and the waves are too small to form a bar at these depths. Some material may be distributed in a thin layer over the offshore area, but no evidence exists to support this theory. Offshore profiles are not accurate enough to detect minor changes reliably.

C.60 Dean (1973) developed a model to calculate the response of normal and storm profiles to a given wave steepness, H_o/L_o. Assuming sand with a median diameter of 0.3 mm, and applying figure 4-31 of the Shore Protection Manual (1973), a winter profile should occur for local seas of height greater than about 2 feet, for northern swell of height greater than 4 feet, and for southern swell of height greater than 6 feet. Assuming this to be correct, the summer southern swell is generally the primary source of onshore movement. Most northern swell is 3 feet or greater when channel shoaling is most rapid. No evidence is available that documents bar-formation at the Harbor entrance with subsequent transport of the material back into the channel.

C.61 The 1977-1978 winter had a severe impact on the California coastline. At Santa Cruz, the east beaches eroded to the base of the cliffs. Much of this erosion occurred during periods of high astronomical tides coincident with the arrival of swell well in excess of 3 feet. Many other beaches along the California coastline suffered similarly. During the high tides, large breakers eroded material from above the -5-foot MLLW elevation and deposited it offshore. Plate 6 documents this profile modification at Santa Cruz. Volumetric calculations indicate that between Black Point and the San Lorenzo River 380,000 cubic yards were eroded from the beach face and 270,000 cubic yards were deposited between MLLW and -20 feet MLLW. The deposited material did not form a bar within the surveyed area, but merely flattened the slope. A net accretion of 9000 cubic yards occurred in the updrift shore segment and a loss of 19,000 cubic yards occurred in the downdrift shore segment. Most of the downdrift loss is postulated to have been transported around Black Point or offshore beyond the -20-foot contour. The record-high precipitation rates for the state in 1978 may have caused an unusually

high deposition of sediment from the San Lorenzo River into the littoral system. However, the harbor did not shoal at unusually high rates during this year. In any event, this anomolous winter should not be used to explain the general shoaling mechanism. Onshore transport due to seasonal profile adjustments is not considered a primary cause of shoaling.

C.62 Rip currents are a secondary mode of onshore-offshore transport. Rip currents have been observed at both the east and the west jetties. A rip current along either jetty can transport material from the beach to the offshore and deposit it where it can later be carried into the channel by a combination of wave agitation and tidal inlet currents. During the summer, a rip current is often generated near the head of the east jetty during a southern swell, but only minor shoaling occurs under these conditions. During the winter, rips are often observed along the west jetty. This is a normal mechanism associated with sand bypassing a saturated groin. Some of the rapid tip-shoal growth in winter and spring may result from the landward return of material moved offshore by these west-beach rip currents; however, this mechanism has been identified as a bypassing of the west jetty and not as onshore transport of bottom material.

C.63 A third mode of onshore transport is the movement of sand in depths greater than 20 feet. Fishermen report sand movements covering their nets in depths as great as 60 feet. Assuming that a bottom velocity of 0.5 feet per second is required to initiate movement, sand motion should be induced by one-foot waves in as much as thirty feet of water. The sand motion would be oscillatory, but superposition of a steady-state bottom current would tend to move the agitated material uni-directionally along the offshore bottom. While considerable sand movement is known to occur offshore, it generally takes the form of a slow migration of ripples in a saltation process. Very little is known about offshore bottom current structure in the study area or even the character of sediments on the offshore bottom. Yancy (1968), however, concluded that movement in the offshore region was primarily the seaward loss of fines to deep water.

C.64 Survey data are not sufficiently reliable off Santa Cruz to detect offshore sand movements. Offshore profiles in depths ranging from 20 to 30 feet show vertical fluctuations of the bottom as large as four feet. These fluctuations appear in the October 1964 soundings and may be measurements of waves rather than actual changes in bottom elevation. The absence of sufficient survey data or adequate quantification of ripple-bed movement renders it difficult to assign a percentage to the portion of shoaling in the channel that is due to onshore transport of material from depths greater than 20 feet. However, it is considered small in comparison with that due to longshore transport.

C.65 Tidal Exchange. Tidal inlets through sandy shorelines may trap and store sediments derived from the littoral environment through the action of tidel-induced currents. Tidal currents are a possible cause of the shoaling of the Santa Cruz Harbor entrance channel. Although no measurements of tidal velocities have been reported, calculated tidal currents for the project channel have a maximum velocity in the order of 0.16 feet per second. Tidal currents become important only after the shoal has formed and a small equilibrium channel through it has been established. Only then are the tidal currents strong enough to transport additional material into the harbor on flood tides. Surge induced currents also would contribute toward shoaling. Evidence of tide-induced or surge-induced current transport would be the growth of a shoal at the inner end of the constricted tidal channel. The absence of such an inner shoal indicates that this mechanism is of minor importance.

C.66 Littoral-Transport Rates. The preceding discussion is a summary of the available data on littoral transport. Order-of-magnitude quantifications of each shoaling process were made based on analysis of the data and application theoretical concepts. A more definitive quantification on which to base the design of structures or development of maintenance programs would be desirable. However, sufficient quantitative data are not available to refine the analysis. Significant variations in quantities may occur from year to year as a result of changes in the bypassing and maintenance procedures, flooding of the San Lorenzo River, and variations in the wave climate.

C.67 Based on the energy-flux calculations, which predict an upper bound for net littoral-transport rates of 488,000 cubic yards per year, and the early impoundment rate at the west jetty fillet and channel shoaling of 285,000 cubic yards per year, the net longshore transport rate is estimated at 300,000 to 500,000 cubic yards per year. The documented high occurred in the winter of 1977-78 when 162,000 cubic yards were transmitted to the east beach by the phased dredging program and about 55,000 cubic yards were transported by the WES plant. The hypothesized quantification of transport into the channel and on to the east is depicted in figure C41. Wind transport into the channel is estimated at 5,000 cubic yards per year over the west jetty and 2,000 cubic yards per year over the east jetty. Transport through voids in stonework is estimated at 20,000 cubic yards per year for the west jetty and 1,000 cubic yards per year for the better sealed east jetty. Most of the material that enters the channel is believed to be that which bypasses the head of the west jetty and forms the tip shoal. The quantity is estimated at 100,000 cubic yards per year. It is estimated that an additional 175,000 to 375,000 cubic yards a year naturally bypass the head of the west jetty, traverse the outer face of the tip shoal, cross the tidal channel and the shoal off the east jetty, and continue on to the east beach. Tidal currents and onshore transport are estimated to account for an additional 2,000 cubic yards of fines entering the channel. This 2,000 cubic yards may be low, but this mechanism is closely related to reversal of transport and bypassing the west jetty. Updrift littoral transport from the east beach around the head of the east jetty via rip currents and wave induced onshore transport is estimated at 20,000 cubic yards per year. The average amount of maintenance dredging deposited on the east beach is estimated at 150,000 cubic yards per year, even though "pay yards" are but two-thirds of this figure. The 175,000 to 375,000 cubic yards that have naturally bypassed, plus the 150,000 cubic yards of maintenance bypassing, minus the 23,000 which re-enter the harbor, leave an additional 2,000 cubic yards which are returned to the offshore from the downdrift beach. The budget is balanced with 300,000 to 500,000 cubic yards going downdrift around Black Point. These figures were developed prior to the phased dredging program. As will be explained in Section D, the more current dredgings with more frequent surveys indicated that the natural-bypassing estimate should be reduced and that the harbor-shoaling estimate should be increased; however, this may be a severe winter shoaling episode and may not reflect long-term trends.

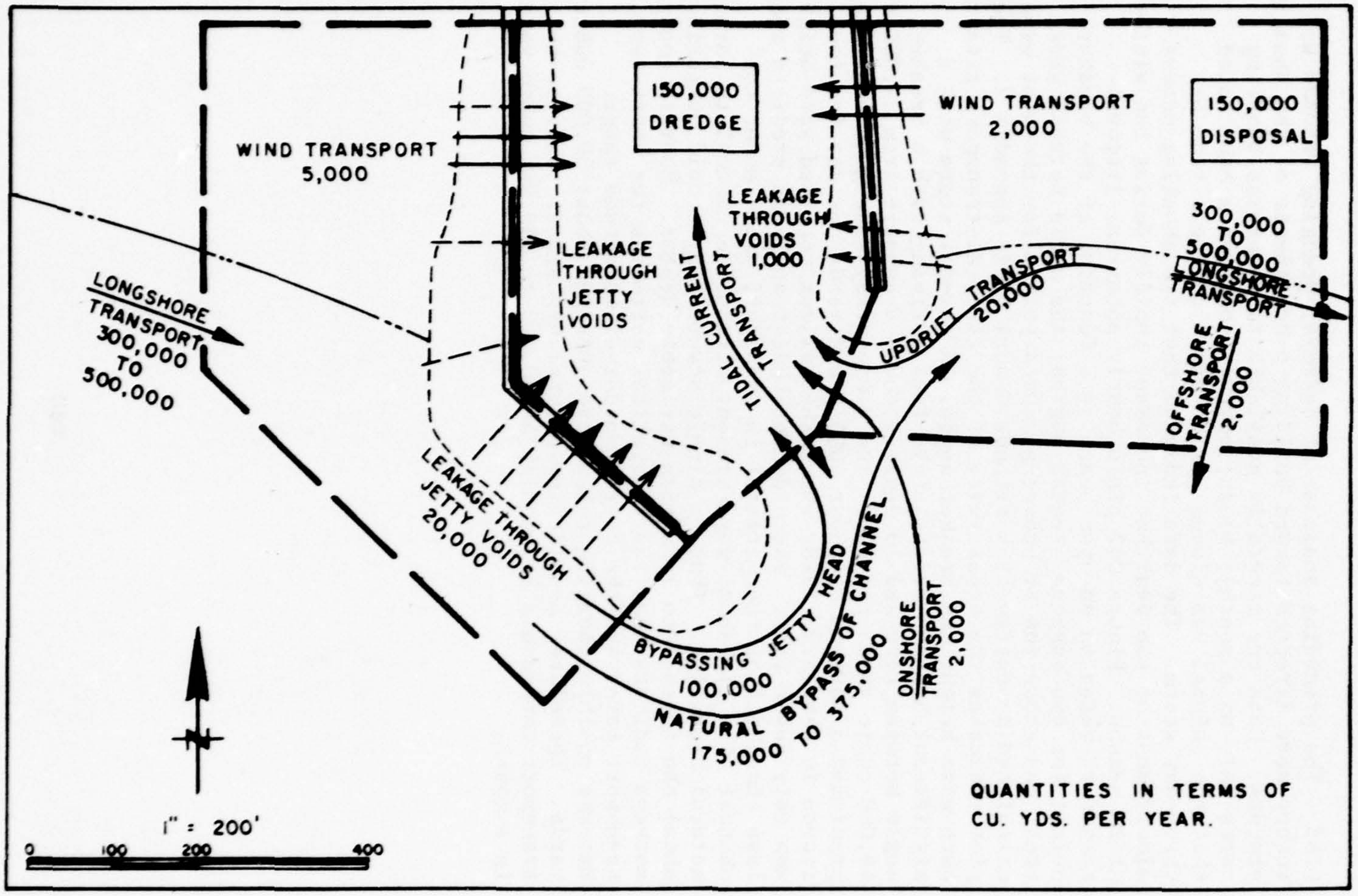


FIGURE C41. SEDIMENT BUDGET

C.68 The preceding analyses of surveys, dredging records and theory were directed toward building a hypothesis of the shoaling process. Data are generally available to determine shoaling rates only on a monthly basis; however, observers have noted that the channel has closed on occasion during a two- or three-day storm. The data indicate that the shoaling occurs slowly most of the year but increases rapidly during the winter storm season. Figure C42 plots hourly potential littoral-transport rates on an open beach as a function of the breaker height for two assumed breaker angles, the angle between wave crest and shoreline at breaking. Data points for the plot were calculated by energy-flux methods described in Appendix 2. The plot indicates the sensitivity of the littoral-transport rate to both wave height and breaker angle. A one-day storm with a significant breaker height of 10 feet arriving with a breaker angle between five and 10 degrees could transport from 24,000 to 48,000 cubic yards per day. The slope-array wave gage analysis predicted a maximum of about 4000 cubic yards per day for two storms in February, 1978, where the maximum recorded wave height was only seven feet. The sediment-budget analysis predicts that less than half of the material in transport would enter the channel if a tip shoal were present across the channel to permit natural bypassing. Thus, a single severe storm could potentially shoal the channel to less than navigable depths. Monthly condition surveys indicate that the theoretical estimates for short-term transport rates are the correct order-of-magnitude range. Extreme monthly shoaling rates have averaged about 30,000 cubic yards. Therefore, considering natural bypassing, the true net transport rate could be as high as 60,000 to 150,000 cubic yards in a month.

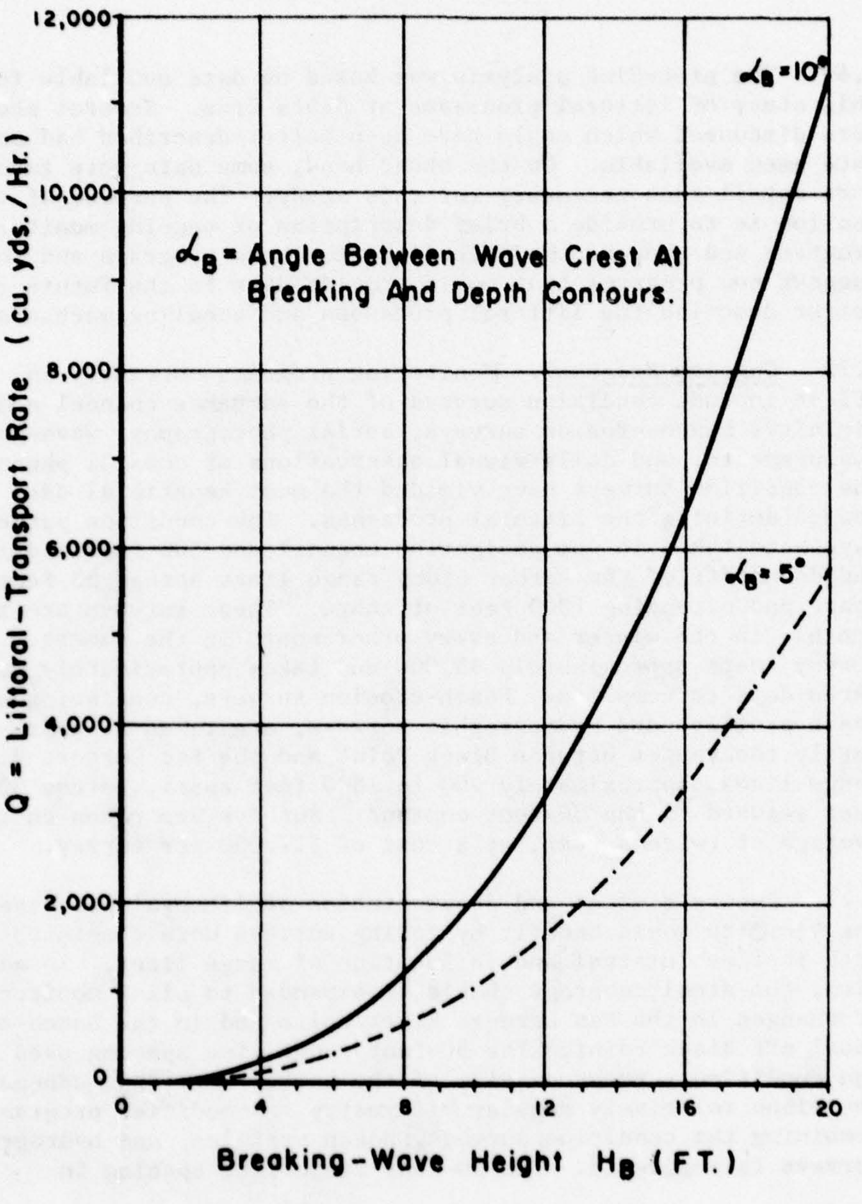


FIGURE C42. HOURLY LITTORAL-TRANSPORT RATES

MONITORING PROGRAMS

C.69 The preceding analysis was based on data available for this study of littoral processes at Santa Cruz. Several phenomena were discussed which could have been better described had more data been available. On the other hand, some data were taken in more detail than necessary for this study. The purpose of this section is to provide a brief description of ongoing monitoring programs and proposed modifications to these programs and to suggest new programs that would provide data in the future to better describe the littoral processes and shoaling mechanisms.

C.70 Ongoing Programs. Monitoring programs currently in effect include condition surveys of the entrance channel and vicinity, beach-erosion surveys, aerial photography, wave-gage measurements, and daily visual observations of coastal phenomena. The condition surveys have yielded the most beneficial data toward defining the littoral processes. The condition surveys have been taken in the navigation channel and 500 feet updrift and downdrift of the harbor along range lines spaced 50 feet apart and extending 1000 feet offshore. These surveys are taken monthly in the winter and every other month in the summer. Each survey costs approximately $5,000 and takes approximately two to three days to complete. Beach-erosion surveys, consisting of beach profiles and hydrographic surveys, are taken at approximately ten ranges between Black Point and the San Lorenzo River. Range lines, approximately 200 to 1000 feet apart, extend 3000 feet seaward to the 30-foot contour. Surveys are taken on the average of twice a year, at a cost of $17,000 per survey.

C.71 Future studies and documentation of littoral processes in the vicinity would benefit by taking surveys more consistently, both in time interval and in location of range lines. In addition, the areal coverage should be expanded to allow monitoring of changes in the San Lorenzo River delta and in the beach and shoal off Black Point. The 50-foot range line spacing used for the condition surveys outside of the harbor more than adequately describes relatively regular bathymetry. A modified program combining the condition surveys, beach profiles, and hydrographic surveys is suggested. The 50-foot range-line spacing in

the channel is required, but the distance between range lines could be increased in areas outside the channel. The first two profiles on either side could be spaced at 300 feet, and beyond them at 500 feet extending westward to the east side of the San Lorenzo River delta and eastward to Black Point. Analyses of previous surveys did not reveal meaningful results for soundings taken in depths greater than 20 feet. Therefore, the profiles need not extend beyond the 25-foot contour, except for infrequent extensions to the 40-foot contour to document long-term changes. The condition-type surveys should be taken on a monthly basis from November through March and in July and September, and should extend only to the first two profiles on each side of the harbor. The phased dredging program may, however, dictate the timing of the surveys. The hydrographic beach-profile surveys should be taken in September and March to document the summer and winter beaches. Provision should be made for a pre- and post-storm survey of the harbor and San Lorenzo River delta to document changes due to a single event. This is most adaptable to the first major storm of a season which follows a scheduled survey.

C.72 An extensive aerial photography monitoring program is currently being conducted. The program comprises twice monthly flights over the area from Natural Bridges Beach to New Brighton State Beach. The photography program costs about $2,500 per flight.

C.73 The photographs are useful in interpolating and extrapolating shoreline configurations; however, the MLLW line cannot be accurately determined from analysis of the photographs because of wave runup and a six-foot tide range. The photography program could be reduced to two or four flights per year and coverage only between Point Santa Cruz and Black Point without significantly decreasing its usefulness. The scale should be about 1:5000.

C.74 An extensive wave-gaging program is being prepared to monitor waves in Monterey Bay. A four-gage array was installed in September 1977 by the State of California and the Army Corps of Engineers, off the west jetty in 20 feet of water. The purpose of the gages is to measure directional wave spectra twice daily. The cost of the program is $14,000 per year per gage. Several other wave gages are being installed in Monterey Bay, in about 20 to 30 feet of water off the piers at Santa Cruz, Capitola,

Sea Cliff, and Monterey, and off the Moss Landing breakwater, as part of an overall wave-measurement program in Monterey Bay. A wave-rider buoy is to be installed in 200 feet of water southwest of Santa Cruz off the Natural Bridges State Beach.

C.75 The overall wave-monitoring program in Monterey Bay is intended to better describe the shoaling problem and coastal processes so that operation and maintenance of coastal harbors may be made more cost-effective. The present slope-array gage system off the Santa Cruz breakwater gives a detailed statistical description of the wave-pressure field. Several years of statistics are required to reliably predict long-term trends in wave climate. The wave gage experienced operational problems in March 1978 during the first winter season of the phased dredging program. Data from the gage should be analyzed in detail to verify sampling techniques. Early verification of sampling techniques and procedures could lead to minor adjustments of the program that would render the results more meaningful.

C.76 Daily visual observations of littoral phenomena are performed under the Littoral Environmental Observations (LEO) program. Presently four stations are in effect: two between the harbor and the San Lorenzo River, and two between the harbor and Black Point. This program is scheduled to continue, at a cost of $2,000 per year. The LEO data collected in the past have been of relatively little value because they were not collected and reported in a consistent manner. Adequate supervision and better-trained observers should eliminate this defect and possibly provide data useful for future planning.

C.77 <u>Suggested Additional Programs</u>. The shoaling process can be documented as a function of incident waves inexpensively and easily by photographs from a time-lapse camera mounted on a bluff or on a pole near the harbor. Frames taken hourly to four times a day over the period of a year would visually document wave action and beach and shoal response. A 15-second burst of movie film once a day at the same time would provide additional information. These simple programs can be a powerful tool in describing shoaling and beach changes.

C.78 The suspected presence of large quantities of sand in the offshore regions represents a large unknown in the littoral regime of the area. Large quantities of material may be in littoral transport offshore. The first step in determining the importance of onshore-offshore transport would be a sampling program to determine the size distribution and depth of unconsolidated bottom sediments out to the 30-foot depth contour. If material is in transport from deep-water off Point Santa Cruz onshore to the beaches in the vicinity of the project site, the size distribution of the beach and offshore material should compare. This should be done to supplement Yancey (1968) in more detail of the project site. The sampling program should also include sieve analysis of the shoaling material in the harbor to compare it with that of the up- and downdrift beaches as well as that found in the river delta. The sampling program could be expanded to include a sand dye-tracer study if the results indicated that materials in the offshore area were of the same gradation as that of the onshore shoaling material.

C.79 The rate at which sediment leaks through and over the jetties can be estimated by dredging a portion of the shoulder lining the perimeter of the channel. By dredging 200-foot-long pockets and taking frequent soundings, the rate of infilling due to various wave episodes could be determined. Careful analysis of the manner in which the pocket fills would lead to a better estimate of the leakage rate through the jetties.

C.80 Other possible monitoring programs include: side-scan sonar, dye-tracer sand studies, and bottom current measurements. Side-scan sonar has been suggested as a possible method of detecting offshore movement of sand waves. Review of profiles indicates that the offshore changes in depths greater than 20 feet have a vertical range of less than about two feet. Side-scan sonar is not suited to detecting movements of slow-moving and small-amplitude sand waves. Dye-tracer sand studies could be implemented to detect the direction of sand movement. Dye-tracer studies are generally expensive and often lead to frustration when the dye patch suddenly disappears. Current meters could be used to measure offshore bottom currents; however, reliable current meters that segregate orbital velocities of waves from uni-directional flow are expensive. Several current meters would be required to plot the current structure with sufficient accuracy to make a meaningful analysis, and the cost of such a program would be prohibitive.

SECTION D

MULTI-YEAR INTERIM DREDGING PROGRAM

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SECTION D

MULTI-YEAR INTERIM DREDGING PROGRAM

BACKGROUND

D.1 General. The preceding sections described the shoaling mechanism and the general dredging practice from 1965 until 1977. The annual dredging procedure had not been satisfactory to the boaters who use the harbor. The channel either had been closed to navigation for the winter or hazardous conditions existed in the entrance channel. Photograph D1 shows a typical winter condition with an eight-foot wave breaking across the tip shoal in the entrance. This condition occurs frequently and has caused broaching of several boats and rendered dredging operations difficult. The tip shoal has created a surf shoal across the entrance channel resulting in conflicts between surfers and boaters. Commercial fishermen who use the harbor as a home port experienced difficulties in some years of not being able to get to sea for the opening of salmon season on April 15.

D.2 This study was undertaken to develop a means of fulfilling the long-range objective of the authorizing document of turning over to the Port District for operation and maintenance an effective sand-bypassing system. The first work item was to develop a multi-year dredging program to shorten the longevity of entrance-clogging shoals, with their attendant navigation hazards, pending development of a long-term solution. A multi-year dredging contract was awarded to the successful bidder, Shellmaker, Incorporated, in November 1977 to prosecute a two-year phased dredging program as developed in a report, "Santa Cruz Harbor, Multi-year Interim Dredging Report," November 1977. The basis of that program development is presented in this section as a new philosophy regarding maintenance dredging at Santa Cruz Harbor. The program that evolved from the study was considerably different from the standardized procedures normally developed by the District, and insufficient time was available to detail it to the satisfaction all parties involved prior to request for bids. Furthermore, the recommended plan required dredging outside the project boundaries, and the WES bypassing experiment was concurrently in progress.

Photograph D1: Wave-Breaking Across Entrance Channel

These factors necessitated some minor procedural changes at the last minute; consequently, the originally conceived plan was prosecuted in concept, but not in full detail. The remainder of this section describes the philosophy of the plan, the basic changes, and the experience gained from the first year of operation. This first-year experience was useful in developing and evaluating alternative long-term solutions and possible modifications of future multi-year interim dredging programs.

D.3 Scope of Multi-Year Program. The basic requirement was to develop an interim, multi-year dredging program that would be more responsive to the needs of boaters by maintaining a navigable channel for a greater part of the year than in the past. The program was to be developed prior to the completion of a detailed study of littoral processes. Its purpose was to reduce dredge-mobilization costs by awarding a contractor a long-term agreement. Part of the problem was to define the optimum length of the multi-year contract.

D.4 Bypassing at Other Harbors. A study of dredging efforts at other small-boat harbors was made to illustrate the relative magnitude of bypassing efforts at Santa Cruz as compared with those at other harbors. The goal of sand bypassing activities at Santa Barbara, Ventura Marina, Channel Islands Harbor, Santa Monica and Oceanside is to bypass all littoral drift and to continuously maintain a safely navigable entrance. Table D1 shows that the annual bypassing at these harbors averages 467 cubic yards per berth. Santa Cruz Harbor has historically bypassed only 110 cubic yards per berth per year. Assuming a gross drift rate of 400,000 cubic yards per year, the 900 berth marina could conceivably require the bypassing of 444 cubic yards per berth to be comparable to the other harbors referenced. On the other hand, the actual bypassing rate required is a function of the local net annual rate of littoral transport, gross rates in each direction, impoundment effects of harbor structures, channel depths and their relation to natural bypassing, etc. The rate per berth is only incidental to these controlling factors. Nonetheless, a review of dredging statistics shows that significantly greater quantities of sand have been bypassed at other harbors than have been programmed for bypassing at Santa Cruz Harbor to date.

TABLE D1.

OTHER SAND BYPASSING EFFORTS

Harbor	No. Berths	Annual Cubic Yards Bypass	Quantity in Cubic Yards per Berth
Santa Barbara	800	300,000	375
Ventura Marina	1,260	700,000	555
Channel Islands	1,540	1,300,000*	422
Oceanside	794	400,000	504
Total	4,390	2,050,000	

Average = 467 cubic yards per berth.

*This effort bypassed two harbors (Channel Islands and Port Hueneme); credit 650,000 C.Y. to each.

DESCRIPTION OF THE PLAN

D.5 General Requirements. A plan was formulated based on the history of shoaling and dredging and on discussions held with the harbor manager and with representatives of the commercial fishermen, the most frequent users of the harbor in the winter season. The strongest plea of the commercial fishermen was to have the dredge available during the winter to clear the channel and particularly to be able to use the channel for the salmon season which starts 15 April. Salmon catches are one of their major sources of income. The fisherman agreed that the harbor entrance should not be used during storms or high-wave episodes and stated that they could accept about a five-day delay after a storm for removal of hazardous shoals. They defined a minimum acceptable depth of 10 feet MLLW and, though not definitive about width, they indicated that a 50-foot channel might be acceptable. However, considering dredge efficiency and problems of breakers in the channel when waves approach at an angle, a 100-foot wide channel was selected as a minimum dimension to be maintained.

D.6 A review of the dredging and accretion histories of Santa Cruz Harbor for the period between 1972-1977 indicates that shoaling rates decrease between April and November and that dredging of the navigation entrance to the full project width and depth in April results in a navigable channel through the summer and the busy boating season. Periodic maintenance dredging would be required between November and April to keep the channel open to navigation. A total sand bypassing program with the existing harbor entrance configuration would not be feasible because there is neither adequate protection to keep a dredge operating during storms nor sufficient storage capacity to create an effective sediment trap. Two conflicting criteria evolve. The channel should be wide and deep enough for safe navigation, yet narrow and shoal enough to maximize the natural bypassing of littoral drift during the winter. A compromise between these conflicting criteria would be to reduce the project depth, at least during the winter. For example, if the navigation channel could be maintained at a depth varying between 10 and 15 feet MLLW over a width of 100 feet, it should be reasonably safe for navigation (except during storm wave episodes) and yet not completely interfere with the movement of littoral drift that naturally bypasses the harbor entrance seaward of the 10-foot depth contour.

D.7 The general requirements for the interim dredging program indicate that a floating hydraulic dredge should be mobilized for effective channel maintenance. The existing WES plant or a dragline operation were not considered suitable to dredge the areas where shoaling occurs. Therefore, an alternative program was developed based on using a floating hydraulic dredge with disposal on the east beach.

D.8 WES Plant. The Waterways Experiment Station has been testing the use of "eductors" as a continuous, fixed bypassing system. Santa Cruz was the site of the third of a series of field tests. During formulation of the interim dredging program, this test was in progress and was scheduled to continue through the winter of the first year. The production of the WES plant varied from zero to about 1,000 cubic yards per day with little consistency of performance, as is required for a reliable system to maintain a harbor. The WES plant was operating on the west beach, bypassing sand to the east beach. This procedure interferes less with channel maintenance operations than with systems using eductors inside the harbor and it should not seriously affect the interim dredging program. However, the WES experimental system was not considered reliable at this stage either as an interim solution or as a part of it. The interim program had to be developed without relying on the WES program and yet, to cause minimal interference with it.

D.9 Length of Contract. While a multi-year contract has some advantages over a one-time contract, the longer the contract the more clearly working conditions must be known to ensure a fair and economical contract without an excessive risk of change orders or claims. On the other hand, few dredges of the 14- to 27-inch class are in operation on the West Coast. More bidders might be attracted by offering a contract that lasts for more than one year. This procedure should reduce mobilization and demobilization costs, which comprise a large percentage of the overall cost of a small, short-term dredging effort.

D.10 A two-year program was selected as being optimum for the first contract. In subsequent years, the time span may be adjusted based on the knowledge gained from the first contract. The contract period selected was from about 1 November 1977 to 1 May 1979. The contractor would be required to have his dredge in

Santa Cruz Harbor mobilized for work by 15 November 1977 and to keep it there either working, or in working condition, until April-May 1978, unless released earlier by the Contracting Officer. He would be free to remove the dredge during the summer, but would be required to have it back on site and ready to work again by 15 November 1978. Any change in equipment would require the prior approval of the Contracing Officer. This arrangement would not save much in mobilization and demobilization costs, as compared to two one-year contracts, but would assure the bidder of work for two years and an opportunity to bid more competitively on the Moss Landing maintenance program scheduled for 1978. This advantage should partially offset the cost of keeping a dredge in Santa Cruz Harbor the entire winter season.

D.11 Dredge Capacity. The dredge must have the capability of dredging the entrance channel to a depth of 10 to 15 feet MLLW within a 5- to 15-day period, but must not be so large as to entail excessive standby charges. Based on experience at Redondo-Malaga, Oceanside, Ventura Marina, and previous efforts at Santa Cruz Harbor, the dredge should be capable of working in 3-foot seas on wires. Examination of figure C36 shows that from December through March, the monthly rate of accretion can exceed 30,000 cubic yards and the daily rate can be as high as 2,000 to 4,000 cubic yards for short intervals. Therefore, the rated capability in calm water should be at least 250 cubic yards per hour, or 4,000 cubic yards per day, based on a 16-hour day with 3,000 feet of discharge line. Considering this minimum production capability and the severe wave climate, it was not likely that a dredge with a discharge line of less than 14 inches could be used. The required capacity gives the contractor incentive to promptly dredge the channel to contract depth, and then revert to standby time with related reductions in crew, and operating costs. Unfortunately, the Corps of Engineers' normal practice of using "Liquidated Damages" covering the costs of inspection crews after completion due-dates to encourage timely performance has little effect on dredging contractors because of their high plant and operating costs. If liquidated damages could include losses incurred by the users of the harbor as well as losses by the Corps, the penalty for late completion could be a more effective tool to ensure prompt and effective dredging.

D.12 The least costly sand-bypassing program at Santa Cruz Harbor schedules the dredging of littoral accretions from the entrance once each year, in March or April. This historical procedure has not been satisfactory to the boaters. The most desirable program is one which would effect a constantly maintained project depth of 20 feet MLLW. This would require almost daily dredging of the gross littoral transport. As a compromise, additional dredging episodes between November and April would maintain the harbor reasonably safe for navigation throughout the year. However, short periods of closure should be expected to allow time for the dredge crew to re-mobilize after a severe winter storm.

D.13 The desired solution falls somewhere between once-a-year dredging and continuous dredging. The adopted plan must, however, assume that presently available equipment will be used. Over-depth dredging or creating a trap did not prove feasible because of the limited capacity of the entrance channel to store sand between the jetties and because of the underlying strata of clay or hardpan. An intangible factor of serious concern was the possibility that any method of channel dredging might divert into the navigation channel a significant portion of the littoral drift that formerly bypassed the harbor naturally, thereby increasing the total amount of sand to be dredged. Inevitably, this will happen to a certain extent; the objective is to minimize this effect. Natural bypassing can be facilitated by limiting winter dredging to reaches shoreward of the 10-foot natural depth of the ocean floor off the entrance except for the final dredging in April.

D.14 Table C8 shows that an estimated average of 43,000 cubic yards accumulates in the control area between April 1 and November 15, after which the channel shoals to depths less than ten feet. The intensity and frequency of storm waves increases after November 15 and an additional 114,000 cubic yards accumulates in the control area to April 1. Hence, an average of 157,000 cubic yards of dredging would be required. This is close to the one-phase 1977 dredging quantity. However, if the material were removed during the winter, additional shoaling of the channel could occur through diversion of natural bypassing. The amount of this additional shoaling can only be determined through experience.

D.15 In a phased dredging program, the contractor would be required to mobilize his crew aperiodically. The cost of these mini-mobilizations could be reduced by having fewer of them. In

order to assure the contractor of reasonably economic dredging operations at such times, each should involve not less than a 30,000- to 50,000-cubic-yard effort. Based on 4,000 cubic yards per work day, this becomes 8 to 13 working days per episode including down time for foul weather and equipment breakdown periods. The proposed dredge schedule shown in table D2 and figure D1 comprises three dredging episodes of 50,000 cubic yards each between November 15 and March 15, plus a final dredging episode in mid-April to maintain the channel open to navigation throughout the summer season. Construction of table D2 was based on the shoaling rates in table C8. Table D2 gives the minimum, average and maximum accretion rates assuming the dredge has a capacity of 4,000 cubic yards per day. The final dredging in April would leave 6,000 to 43,000 cubic yards to dredge, plus an unknown quantity of sand that might be drawn into the entrance by diversion of natural bypassing.

D.16 Dredge size and efficiency should not basically affect the concept of three dredging episodes (phases) of 30,000 to 50,000 cubic yards each, plus a final phase in April. A small-capacity dredge would take longer to clear the channel and would have fewer standby costs than a larger dredge, but a larger dredge would open the channel sooner and lose less operating time because of high wave action. The larger dredge, by clearing the channel quickly, would pump less sand because less time would be available for more sand to enter the channel during dredging operations. However, after each phase, more sand which otherwise would naturally bypass the harbor could be trapped. Mobilization, demobilization, and standby costs would, of course, be higher with a larger dredge.

D.17 The littoral transport rate fluctuates from year to year, and the dredging requirements should be flexible enough to accommodate these fluctuations. Figure D1 illustrates the potential effects that the dredging program could have on shoaling volume, assuming that 50,000 cubic yards would be dredged per phase and that the harbor would shoal at the maximum observed rate. A net amount of 20,000 cubic yards could accrete by April 10 at the start of the final dredging episode. In contrast, assuming that only 30,000 cubic yards would be dredged per phase in conjunction with the minimum assumed accretion rates, a net amount of 43,000 cubic yards could accrete by April 10.

Table D2.

PROPOSED ANNUAL DREDGE SCHEDULE
(Quantities in 1000 Cubic Yards)*

Date	Action	Minimum Accretion Q Dredged	Q Accreted	Q Accum.	Average Accretion Q Dredged	Q Accreted	Q Accum.	Maximum Accretion Q Dredged	Q Accreted	Q Accum.
	Prior to Nov. 15			29			41			52
Nov. 15	Start Phase I Dredge									
Nov. 30	Complete Phase I Dredge	- 30	+ 6	+ 5	- 50	+ 7	- 2	- 50	+ 8	10
Dec. 1	Start Standby Time									
Dec. 31	Complete Standby Time		+33	+38		+25	+23		+29	39
Jan. 1	Start Phase II Dredge									
Jan. 15	Complete Phase II Dredge	- 30	+ 8	+16	- 50	+13	-14	- 50	+14	+ 3
Jan. 16	Start Standby Time									
Feb. 28	Complete Standby Time		+34	+50		+43	+29		+42	+45
Mar. 1	Start Phase III Dredge									
Mar. 15	Complete Phase III Dredge	- 30	+12	+32	- 50	+14	- 7	- 50	+12	+ 7
Mar. 16	Start Standby Time									
Apr. 9	Complete Standby Time		+11	+43		+13	+ 6		+13	+20
Apr. 10	Start Final Dredge									
Apr. 23	Finish Final Dredge	- 50**	+ 1	- 6	- 20**	+ 2	-12	- 20**	+ 3	+ 3
	ANNUAL NET	-140			-170			-170		

*Quantities are approximate and based upon accretion curves of figure 7.

**This does not include possible additional littoral drift drawn into area during Phase I, II or III dredging. A change order may be required if this addition is substantial. This quantity assumes dredging is to Project Dimension.

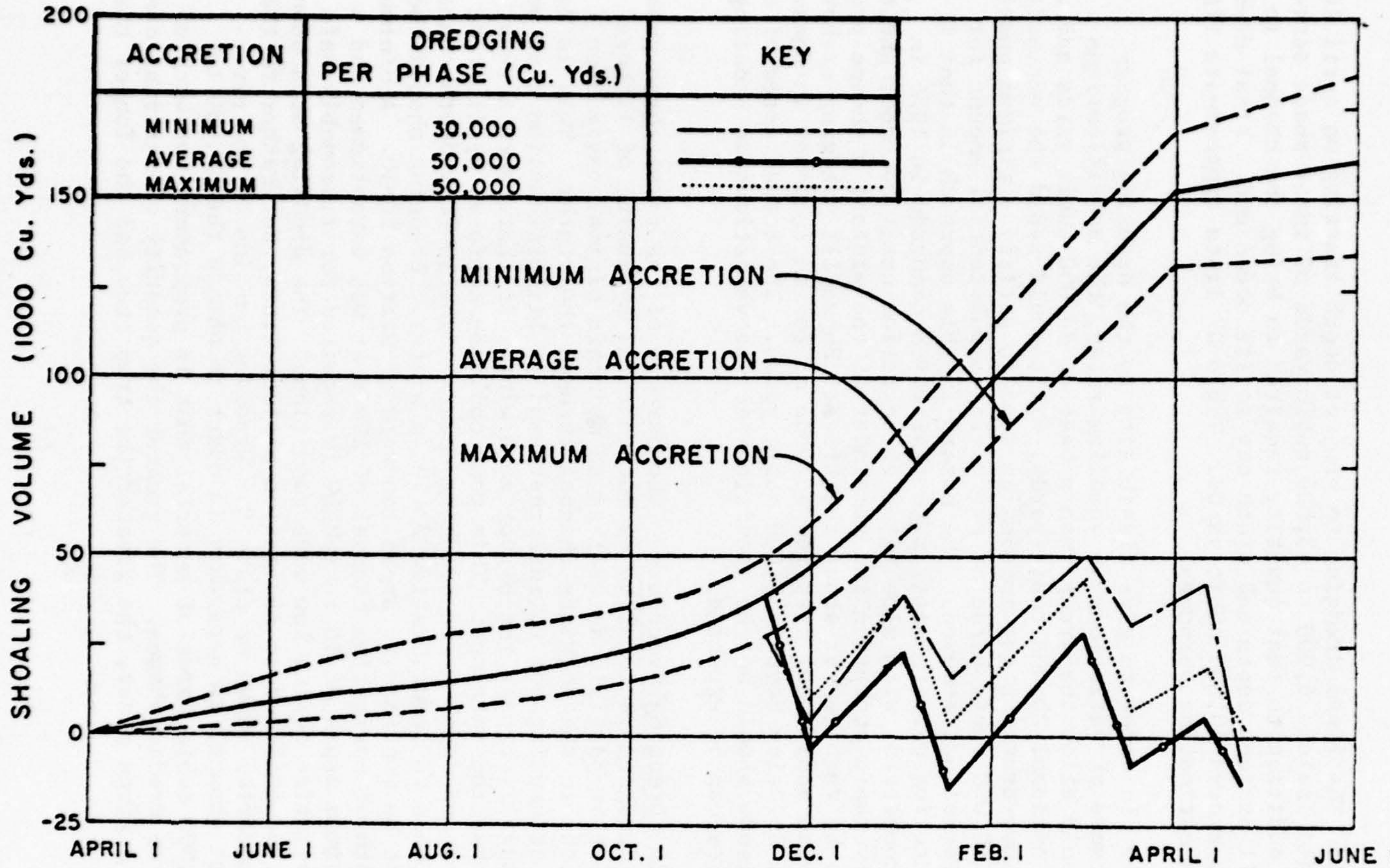

FIGURE D1. AVERAGE, MAXIMUM AND MINIMUM CUMULATIVE RATE OF ACCRETION AS MODIFIED BY PROPOSED DREDGE PROGRAM.

D.18 The final dredging to project depth starting on April 10 could include 6,000 to 43,000 cubic yards of inter-phase accretion in addition to that quantity required to bring the channel to full project depth and width out to its ocean end. Final dredging may exceed 50,000 cubic yards. Table D2 lists approximate figures for estimating purposes.

D.19 In order to give flexibility to the dredging program because of variations in shoaling rates, the specifications should allow the dredgerman a base of 30,000 cubic yards and with an optional 20,000 cubic yards. This could avoid the necessity of overdredging the impounding area to satisfy a minimum quantity of 50,000 cubic yards or overpaying a substantial amount for material not removed. The hazard in this approach is that if an accretion pattern similar to that which occurred in 1976 is encountered, with slow rates of accretion until December and then very rapid accretion the remainder of the winter, a change order may be required at about the time of Phase III dredging either for an additional dredging episode or for an increase in Phase III dredging beyond 50,000 cubic yards. The actual dates of each episode should not be specified because variations in shoaling rates can be expected.

D.20 Dredging Procedure. The purpose of the first three phases of dredging is to allow as much natural bypassing of littoral drift as possible as well as to maintain minimal navigation depths in the entrance channel through the winter. This is done by attempting not to interrupt sand in littoral motion that would naturally bypass the harbor each winter if there were a shoal across the entrance. This goal could be achieved by limiting the area and depth where dredging occurs. The 10-foot depth contour extends to about Station 22+50 in winter. Dredging should therefore be confined to areas landward of Station 22+50. A minimum width of navigation channel of 100 feet was established and a minimum depth of 10 feet MLLW is required for reasonably safe navigation during low wave conditions. The dredging area should not be confined to navigation project lines, but rather to the boundaries shown on plate 9. Dredging outside of the project channel is necessary in order to obtain the 30,000 to 50,000 cubic yards of material that is programmed for each of the three dredge phases. The greater the quantity of material dredged on a given phase, the greater the trap area and the longer the

period of time the harbor would remain open. A winter project depth of 14 feet MLLW with allowable overdepth dredging of 2 feet is recommended. An average channel depth of 15 feet should result. If the quantity to be dredged does not fall within the desired 30,000 to 50,000 cubic yard range, then the taking area should be deepened, concentrating first in areas inshore of Station 19 to get the minimum quantity of material. This change should be directed by the Contracting Officer and designed to take the maximum amount of sand while minimizing the diversion into the channel of littoral drift that would otherwise naturally bypass the channel.

D.21 In order to provide reliable quantity payments and reduce the need for costly surveys, it is suggested that the contractor be permitted to establish adequate range or station lines, both parallel and perpendicular to the centerline of the channel. Pay yardage estimates may be computed using the following procedure:

1. A standard pre-dredging survey would be the responsibility of the contractor.

2. Progress payment measurements could be made by the Contracting Officer using leadline soundings from the forward portion of the dredge barge and averaging the soundings.

3. Side slope payment would be accepted as an assumed one vertical on three horizontal slope from the limit of swing in each direction.

4. The standard ±15% "Variation in Estimated Quantities", as stated in the Special Provisions of standard contracts, would apply to these three phases of dredging.

D.22 In planning the program, a fixed time schedule, approximately as shown in table D2, should be assumed. However, the program should have enough flexibility so that if a sudden shoaling to depths of less than 10 feet occurs before a dredging phase is scheduled to begin, the contractor could be ordered to start that phase sooner. The specifications should provide for such an eventuality, assuring the contractor adequate advance notice (say five days) for each early start. With these provisions, it is anticipated that minimum depths of 10 to 16 feet could be

maintained except during unusually severe storms when the channel would become hazardous to navigation and could not be used in any event. Such a storm might last for three to five days, and the channel would be closed for the additional time required to mobilize the dredge and remove the shoal. A small dredge may have occasional difficulty in working during the mid-winter season. If so, and at such times, the specifications should allow the Contracting Officer to reduce the amount of dredging required and to permit removal of material from inside the jetties rather than from the more exposed portions of the taking area.

D.23 Final Dredging (April). Three imponderables necessitate flexibility in quantity as to the amount of dredging required in the final phase.

1. The first three dredging phases will be done while littoral transport is very active. Observation of the W.E.S. eductor experiment between July 1976 and January 1977 indicate that the holes created may have caused an accelerated movement of littoral sand into these depressions. Actual experience will be the only reliable basis for estimating this shoaling factor, which could increase the shoaling rate by as much as 100 percent.

2. The program is based on the normal low-flow regimen of the San Lorenzo River. In the event of a major flood, sand and other debris deposited by the river would be carried to the east and have a major impact on the condition of the harbor entrance. A change order could be required.

3. Littoral transport rates greater or less than those experienced in the last five years may occur. In the five-year period from 1972 to 1977, annual rates did not vary more than 20% from the average. Therefore, a cumulative total of ±30,000 cubic yards should provide a reasonable contingency.

D.24 The bid item for the final dredging starting on April 10th should comprise a base of 50,000 cubic yards with an optional additive of 30,000 cubic yards. This dredging, as previously stated, would be to full project width and depth providing a usable navigation channel during the absence of the dredge from April 30 to November 15. Specification for the final dredging eipsode should stipulate that a minimum depth of 10 feet MLLW

throughout the project channel must be achieved by April 15 in order to open the harbor for the fishing season. The remainder of the channel could be dredged later.

D.25 <u>Disposal of Dredged Material</u>. Requirements for disposal of dredged material on the east beach would be the same as those specified for previous maintenance efforts. Material was customarily transported by pipeline about 1000 feet east of the channel and deposited on the beach over the berm and into the wave uprush zone. The primary disposal area for the phased program should be as far east of the channel as possible. This would encourage movement of sand around Black Point and reduce the quantity of sand re-entering the channel from around the east jetty during periods of southern swell and local seas from the southeast. The contractor could be given the option of burying the discharge line along the beach or, if approved by local authorities, using a surface line and removing it from May through October.

D.26 Dredged material should be deposited a minimum distance of 1,800 feet downdrift of the east jetty from November to March, and this distance should be increased to 2,200 feet in March and April. The normal method of using a Y in the discharge line should provide the necessary flexibility for depositing sand uniformly along about 700 feet of beach. The east beach opposite Schwann Lagoon normally recedes several hundred feet in February. When this occurs, the discharge could be diverted by a Y to renourish the beach in this area. A diffuser at the end of the discharge line need not be required unless the pipe is larger than 18 inches. However, sand should be injected directly into the surf zone to facilitate its movement around Black Point.

D.27 The history of the beaches east of the jetty indicates that the sand normally moves on downcoast around Black Point. It would be desirable to enhance this downcoast progression by scheduling dredging episodes early in the year when waves are generally from the northwest and, therefore, more conducive to passing the material beyond Black Point. During later months when the waves have a more southerly direction, extension of the discharge line over, through, or around Black Point would reduce upcoast migration of sand into Santa Cruz Harbor. The cost of routing the discharge in such a manner would be excessive.

D.28 Coordination with the City of Santa Cruz, State Beaches and Parks, the California Coastal Conservation Commission and other agencies is required to establish the disposal site in a location that will best satisfy requirements of all agencies involved.

D.29 Standby Time. For purposes of this project, "standby time" is defined as the time when the dredge and its support equipment are required to stay in the harbor and be in working condition. It does not include time for repairs or delays due to rough weather. The dredge would essentially be in caretaker status, and the contractor could expect at least a five-day warning before being required to initiate a new phase of dredging. The Harbor Manager has informally indicated two areas where the dredge could be moored during standby periods. He has also indicated a location where the dredge could stay between May and November if the contractor elects not to remove it from the area. If standby time is not shown as a bid item, the contractor must increase his unit price for dredging, based on his own estimate of standby-time losses. Inclusion of the "standby time" bid item removes this element of guesswork from the bidding process.

D.30 Bidding Schedule. Table D3 presents the recommended bid schedule. While approximately two weeks are allowed for each dredging episode, actual standby time will vary according to actual length of dredging episode. A daily standby rate is used to encourage the contractor to shorten his dredging time, which would be an operational benefit to the harbor. If a lump sum standby item were used, a proportionate reduction of standby time would have to be given for the time any dredging operation was slow and overran the start of a standby period.

D.31 Pre-Bid Conference. This proposed maintenance procedure is unusual. The dredging fraternity must thoroughly understand the goals and anticipate problems to be encountered. A pre-bid conference not only would encourage and enlighten potential bidders, but their feedback frequently helps to improve the wording of the final specifications to the advantage of all concerned.

Table D3.

PROPOSED BIDDING SCHEDULE

MAINTENANCE DREDGING
SANTA CRUZ HARBOR,
SANTA CRUZ COUNTY, CALIFORNIA

Item No.	Description	Estimated Quantity	Unit	Unit Price	Estimated Amount
1977 - 1978					
1.	Mobilization and Demobilization	1	Job	L.S.	$____
2.	Phase I Dredging				
	a. First 30,000 C.Y.	30,000	C.Y.	$____	$____
	b. Over 30,000 C.Y.	20,000	C.Y.		
3.	Phase II Dredging				
	a. First 30,000 C.Y.	30,000	C.Y.	____	____
	b. Over 30,000 C.Y.	20,000	C.Y.	____	____
4.	Phase III Dredging				
	a. First 30,000 C.Y.	30,000	C.Y.	____	____
	b. Over 30,000 C.Y.	20,000	C.Y.	____	____
5.	Standby	100	Days	____	____
6.	1978 Final Seasonal Dredging				
	a. First 50,000 C.Y.	50,000	C.Y.	____	____
	b. Over 50,000 C.Y.	30,000	C.Y.	____	____
1978 - 1979					
7.	Phase I Dredging				
	a. First 30,000 C.Y.	30,000	C.Y.	____	____
	b. Over 30,000 C.Y.	20,000	C.Y.	____	____
8.	Phase II Dredging				
	a. First 30,000 C.Y.	30,000	C.Y.	____	____
	b. Over 30,000 C.Y.	20,000	C.Y.	____	____
9.	Phase III Dredging				
	a. First 30,000 C.Y.	30,000	C.Y.	____	____
	b. Over 30,000 C.Y.	20,000	C.Y.	____	____
10.	Standby	100	Days	____	____
11.	1978 Final Seasonal Dredging				
	a. First 50,000 C.Y.	50,000	C.Y.	____	____
	b. Over 30,000 C.Y.	20,000	C.Y.	____	____

EVALUATION OF 1977-1978 DREDGING PROGRAM

D.32 History. The winter season in 1977-1978 was the first year of the two year interim dredging program. Because of difficulties encountered in preparing plans and specifications for bids, implementation of the first phase of the dredging contract was delayed one month. The dredging contract was awarded to Shellmaker, Inc., on November 2, 1978, and the first phase of dredging began on December 19, 1978. The Shellmaker dredge was a 12-inch plant with a pumping capaçity of 300 to 400 cubic yards per hour. The dredge was equiped to work on wires. Two major modifications were made on the dredge to render it more seaworthy. The floatation was increased by adding a collar around the hull, resulting in three feet of additional freeboard. The 27-foot long ladder was extended to fifty feet to flatten the angle of the ladder while dredging. The ladder was damaged when subjected to a large bending moment during the first phase while dredging on the south side of the harbor. The ladder was repaired and shortened to 40 feet for subsequent phases. During the three scheduled phases, the one special phase, and the final phase, 162,000 cubic pay yards were dredged. The WES plant pumped an additional 55,000 cubic yards from August 1977 to March 1978.

D.33 Changes in the Dredging Plan. The final dredging plan and specifications did not conform fully to the plan described in the "Santa Cruz Multi-Year Interim Dredging Report," 1977. The boundaries of the taking areas were modified as shown in plate 9 by the dotted line. The primary changes were to limit the taking area inside the protected area to the project channel width and to extend the taking area outside the protection of the jetty an additional 150 feet to station 24+00. The priorities for removal of material were as shown in plate 9. The change in boundaries and priorities were made for several reasons. One involved the difficulties encountered in changing the project dimensions. For example, widening the channel inside the protected area could have interfered with the WES experiment. The side shoals that border the channel were postulated to reduce surge in the harbor; it was considered desirable to leave them in tact. These changes in the boundaries and priorities altered, in part, the principle of operation of the plan as originally conceived, which was to remove as much material as possible

within the protected area of the west jetty so as to maximize the storage potential while minimizing interference with the natural bypassing. The implemented plan was focused more on intercepting sand outside of the protected areas and channel. Considerably less storage area was created within the harbor where shoaling interferes with navigation. Insufficient time was allowed to predict the possible impacts of these changes on the dredging phases.

D.34 The primary disposal site 2500 feet down drift of the east jetty could not be used because of the difficulty of obtaining permits and easements on short notice. Therefore, the disposal area was at the secondary site 1000 feet down drift of the east jetty.

D.35 <u>Dredging Experience</u>. The quantities dredged during the first year of the multi-year interim dredging program are listed in table D4. A volumetric analysis of the control area shown on plate 10 and used in developing the interim dredging program was based on the condition surveys that were taken in the 1977-1978 winter. The results are plotted in figure D2. Figure D3 also includes quantities removed by the WES experiment and quantities removed by maintenance dredging. The shoaling rate within the control area actually decreased between July and mid-October, and the final survey revealed 10,000 cubic yards less material in the control area than in April, 1977. This is partially attributed to the WES plant, which removed approximately 20,000 cubic yards from the west beach and from areas within the harbor during the periods. Because the first phase was delayed one month, the amount of material that shoaled in the control area was about one month behind schedule. The first phase lasted 12 days and removed 28,000 pay yards. During the dredging operation, storm waves damaged the dredge. The inspector was taking soundings as work progressed, and pay yardage was based on his soundings in lieu of the customary post-dredging survey. Immediately after completion of this dredging a series of storms and large waves hit the Santa Cruz area, and the harbor shoaled rapidly. The second phase was initiated near the end of January 1978, and two additional phases plus the final phase were completed during the first winter. The full impact of the program during the winter of 1977-78 is difficult to assess, but it did allow navigation through the entrance during most of the winter, a feature that was not

TABLE D4

DREDGING QUANTITIES 1977-1978

DATE	WES		SHELLMAKER, INC.		COMMENTS
	Channel (Cu. Yds.)	West Beach (Cu. Yds.)	Pay Yardage (Cu. Yds.)	Contractor's Estimate (Cu. Yds.)	
June 15					Completion of 1977 Dredging Episode
1 July 77-31 July 77	2,131				
1 Aug. 77-31 Aug. 77	1,200				
1 Sept. 77-30 Sept. 77	2,000				
1 Oct. 77-31 Oct. 77	4,730	4,716			
1 Nov. 77-30 Nov. 77	13,000	12,510			
1 Dec. 77-31 Dec. 77	8,915				
19 Dec. 77-31 Dec. 77			28,328	32,526	Phase I
1 Jan. 78-20 Jan. 78	2,625				
27 Jan. 78-3 Feb. 78			35,517	42,258	Phase II
15 Feb. 78-28 Feb. 78	4,336				
17 Feb. 78-21 Feb. 78			23,875	33,500	Special Dredging
1 March 78-10 March 78	988				
6 March 78-17 March 78			18,392	20,492	Phase III
12 May 78-10 June 78			55,758	67,000	Final
TOTAL	39,925	17,226	161,870	195,776	

1. Total WES 57,151
2. Total Channel Dredging (WES & Pay Yardage) 201,795
3. Total Dredging (WES & Contractor's Estimate) 235,701

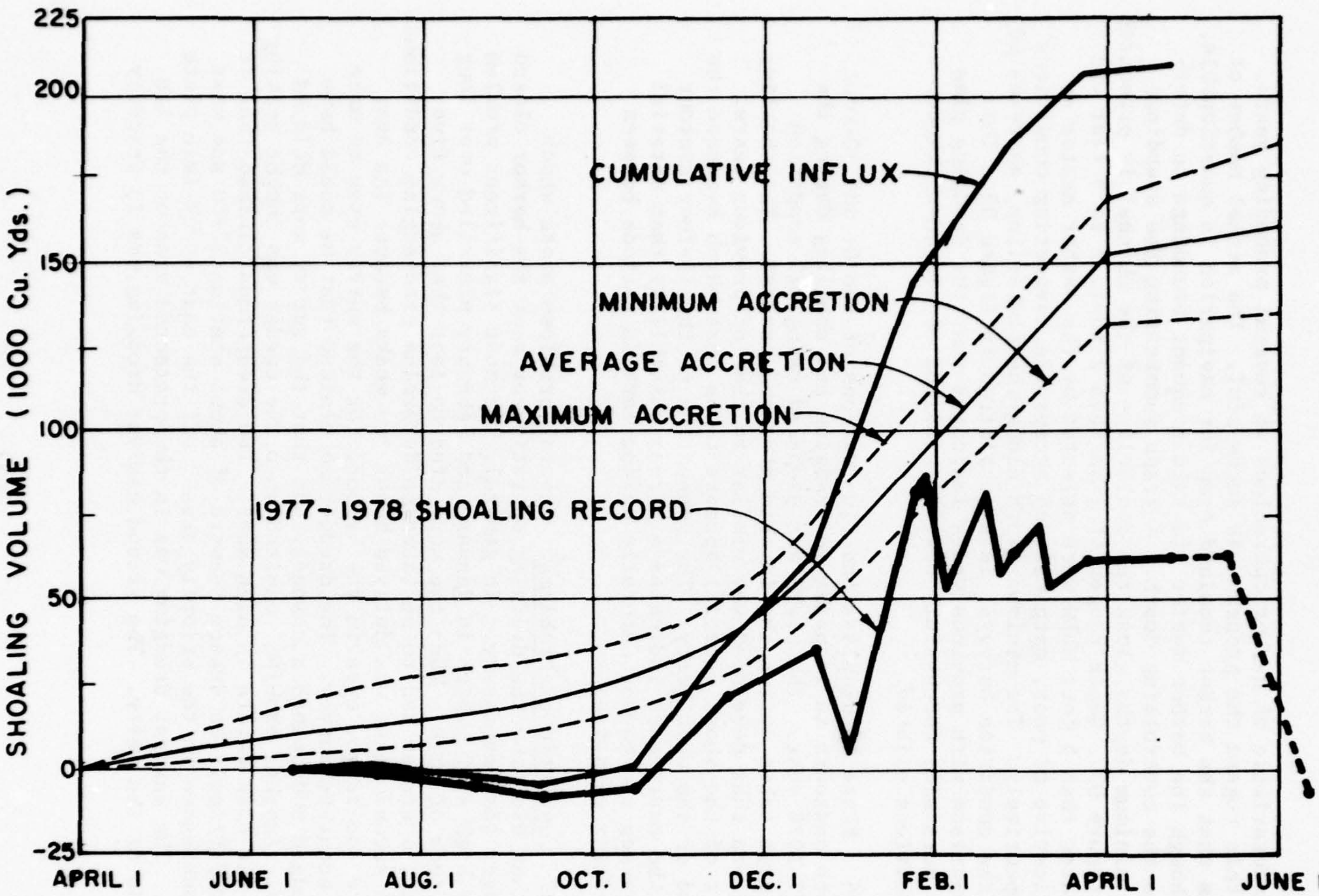

Figure D2: Cumulative Accretion and Dredging History of 1977-1978 Interim Dredging Episode

characteristic of harbor operations in several preceding years. In this regard the program was successful. The actual number of days that the harbor remained open for navigation is questionable, although the harbor master did take frequent soundings to determine the controlling depth. A graph summarizing the soundings and minimum depths along the centerline of the channel is presented in figure D3. Depth ranges of less than 2 feet, 2 to 5 feet and greater than 5 feet MLLW were selected by the harbor master as indicative of poor, marginal and acceptable navigating conditions respectively. The minimum depth along the centerline, as revealed by the condition surveys, is also plotted in figure D3. The comparison with previous years indicates that the dredging plan was partially successful; however, periods of closed-entrance conditions existed.

D.36 Plate 10 displays the fluctuations of the 0- and 10-foot depth contours in response to dredging and shoaling during the 1977-1978 year. The plate was prepared using all condition surveys taken during the interim dredging program. The shoaling pattern that developed was similar to that of previous years. Most of the shoal material appears to be that which bypassed the head of the west jetty. The connection of the 10-foot contour to the east beach indicates a strong possibility that material bypasses the harbor naturally during certain periods between dredging episodes.

D.37 Operational Problems. Several problems arose which either hindered the dredging operation or kept the harbor closed longer than necessary. In general, the most significant problem was high surf, which in January and February prevailed over long periods of time. When the surf died to less than about five feet, a survey had to be taken to determine predredging conditions. The second phase was delayed about two weeks because the seas were too rough, even in the harbor, for the survey crew to make an accurate survey. The dredgerman claimed that he could have dredged and opened a channel, but that the survey work delayed him. Another problem resulted when the dredge was caught working in the open waters in high surf. The dredgerman claimed that it was difficult to dredge seaward of about station 22+00 and most troublesome in the priority area 3 off the east beach (see plate 9). The easiest dredging was in the protected area on the lee side of the jetty. The second easiest dredging was in priority

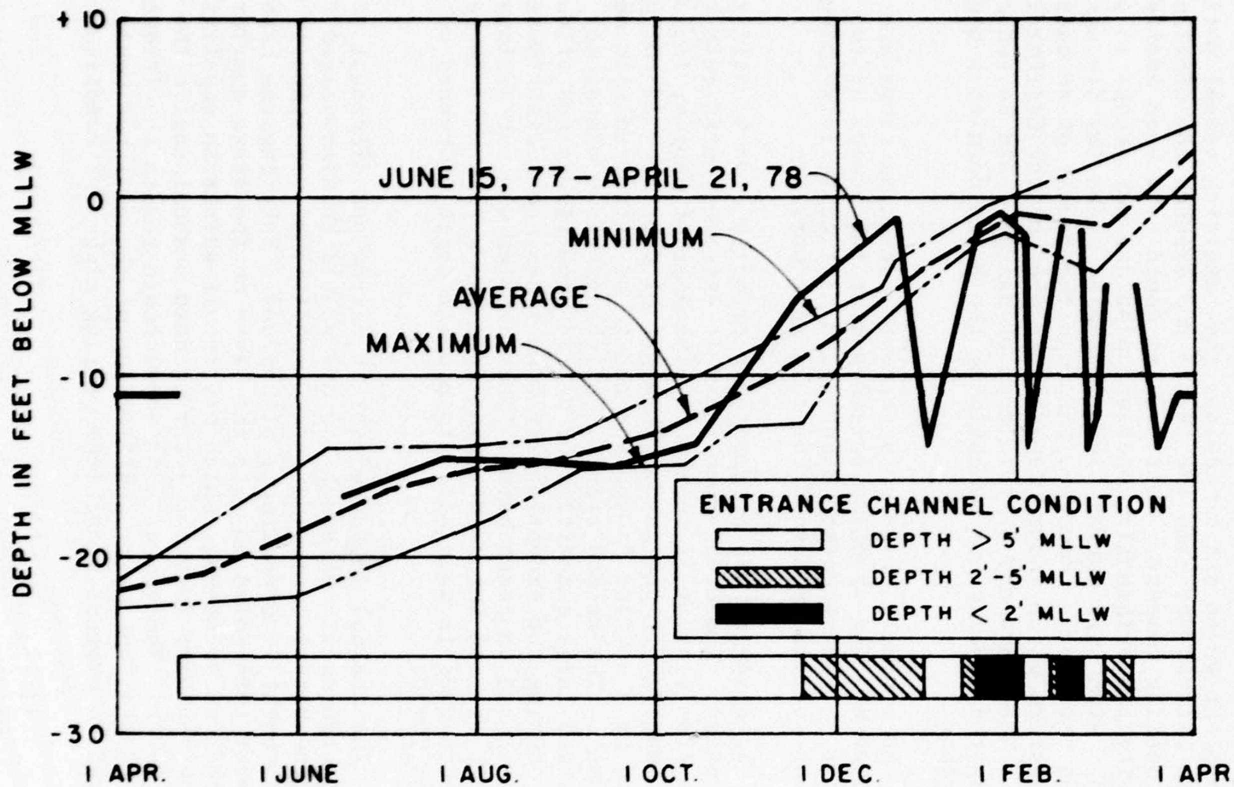

FIGURE D3: CHANGES IN CONTROLING DEPTH ALONG CENTERLINE AND HARBORMASTERS EVALUATION

area 2 between station 22+00 and 22+50. A dredge can operate in higher seas heading directly into them or quartering with them. but when the waves are broadside, the resulting vessel motion induces lateral stresses the ladder and exposes the pipeline to large bending moments. This problem could have been avoided by following the originally conceived plan and not dredge in priority area 3. High surf during high tides also broke the discharge line on the beach. However, this was the result of an unusually severe storm which caused damage over most of the California coast. The frequency of this type of damage could be reduced by burying the pipeline or by building the beach berm to a higher elevation.

D.38 As a result of union agreements, dredging crews are hired by the week. The dredging episodes last for a week or two at the most. The dredgerman experienced difficulty in keeping crews on station as needed for this spot work.

D.39 No dredging was attempted in priority area 4 until the final phase. The dredgerman found it extremely difficult to dredge from station 11 to about 16 because of debris, including logs, embedded in the sand. This mat of debris could not be removed by a suction dredge and had to be extracted with mechanical equipment. The record rainfall of the previous winter may have discharged large quantities of debris from rivers and allowed waves, during the exceptionally high tides, to redistribute debris stored on beach berms. This problem was not serious in prior years, and future dredging episodes may be more successful if the channel is maintained to project depth shoreward of station 20+50.

D.40 The channel in the winter is narrow and difficult to use during high surf. Although boaters should be discouraged from using the harbor during dangerous conditions, occasionally they become caught in undesirable situations. Entering the harbor, they must turn so as to take the waves on the stern quarter. This induces broaching, which may result either in capsizing or driving the boat into the jetty or onto a shoal inside the dogleg area. Removal of this shoal could reduce the frequency of accidents, but major modification of the entrance would be required to prevent broachings during high-surf conditions.

RECOMMENDATIONS

D.41 The experiences gained during the first year of the interim dredging program should be utilized to develop better programs for the future. The following program roughly outlines the recommended procedure that would result in more cost-effective maintenance dredging.

D.42 Dredge Capacity. The pumping capacity of the dredge is not the primary concern. The dredge should be seaworthy enough to work in at least three-foot seas or 5-foot, long period swell on wires. The pumping rate should be at least 4000 cubic yards per day. The modified, 12-inch dredge proved marginally acceptable.

D.43 Dredging Procedure. The recommended dredging boundaries, priorities and depth are shown in figure D4. This procedure closely follows the logic of the program guidelines in paragraphs D1 through D31. Priority 1 is to dredge the main channel to -20 feet MLLW between stations 17+50 and 20+50 and to -14 feet MLLW between stations 20+50 and 22+50. This is the primary area that shoals and hinders navigation. This procedure attempts to establish a good balance between increasing trap capacity and minimizing the disturbance of natural bypassing. Priority 2 is to dredge a trap to some depth between 14 and 20-feet off the west jetty. The trap should not extend seaward of the area shown because that would tend to reduce natural bypassing and increase dredging quantities. Priority 3 is to create a trap in the area of channel where the dredge is protected by the west jetty. This area is outside the project channel but is the key to increasing trap area in protected areas where natural bypassing is not affected by dredging. If done within the boundaries shown, a sufficient sand shoulder should remain on the seaward side to attenuate wave energy entering the harbor. Priority 4 is similar to priority 3 but on the other side of the channel. Priority 5 is to maintain the channel to 20-foot project depth from stations 14 to 17+50. These priorities are assigned for the phased episodes. The goal is to collect a minimum volume of sand and to create as large a trap as possible to depths specified for each priority area.

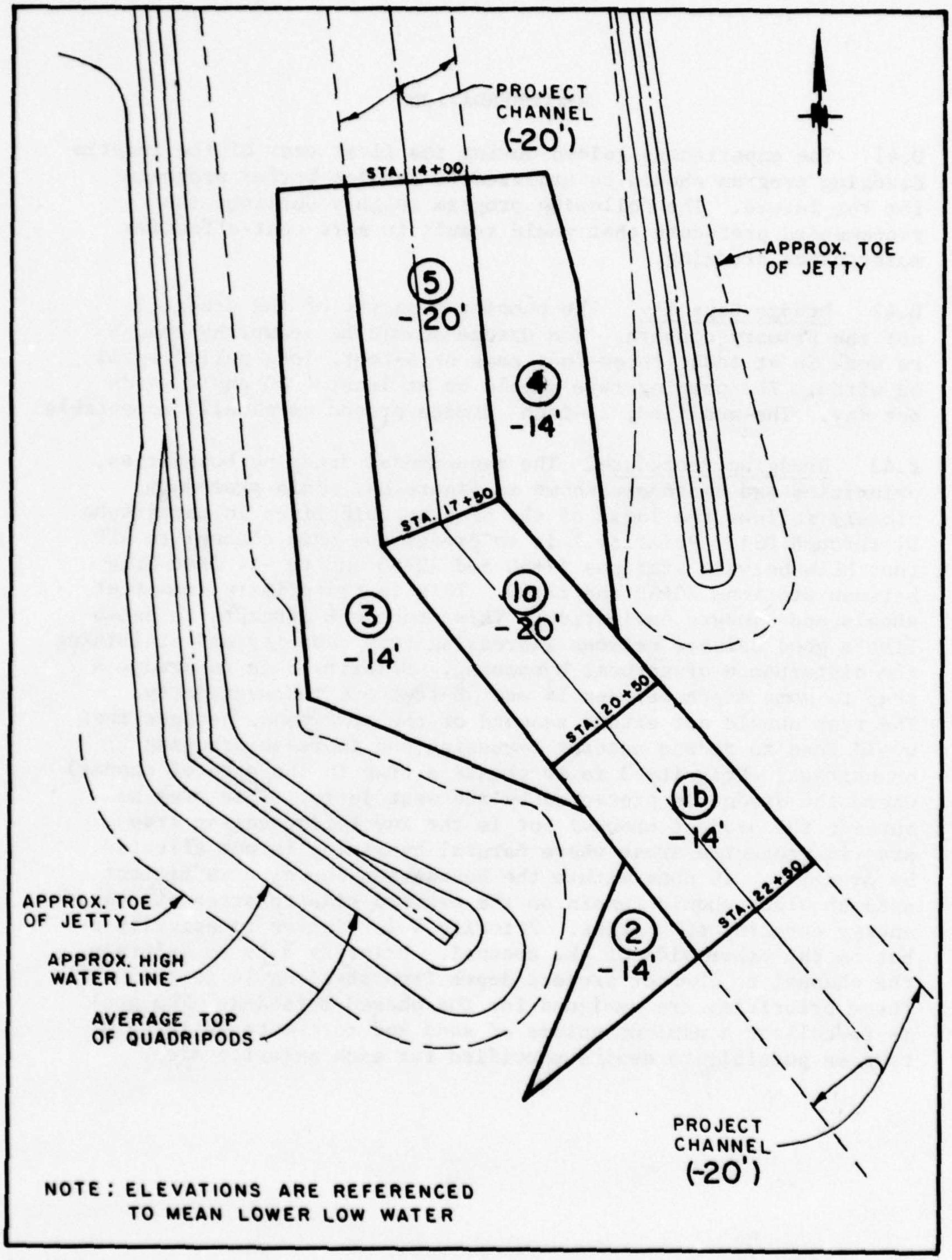

Figure D4: Recommended Dredging Boundaries and Procedures

D.44 Disposal. The disposal areas and procedures outlined in paragraphs D.25 through D.28 are recommended.

D.45 Stand-by Time. Inclusion of an item for stand-by time as described in paragraph D.24 could result in a contract more fair and reasonable to both the dredgerman and Government.

D.46 Method of Payment. The standard pre-dredging and post-dredging survey procedures are not satisfactory for this type of phased dredging. The method used in the 1977-1978 dredging in which the inspector takes soundings from the dredge should be continued. This assures the dredgerman that he will be paid for material removed, even in the event a storm suddenly causes a suspension of the operation. Another possibility would be to include a bid item establishing an hourly rate for dredging during times when quantity measurement is not practical. This would give the contracting officer the option of removing shoal material immediately after a shoal is formed without having to wait for a pre-dredging survey. This hourly rate could apply to dredging in priority area 2 where sand may enter the trap as fast as it is removed. This item should receive only minor consideration in bid selection, but it would provide a method of achieving more efficient use of the dredge and its crew. It could result in a much greater number of days that the channel would remain open.

D.47 Schedule. The dredge should be mobilized by November 15, and the first phase should be implemented when the depth becomes less than 10 feet as specified in the original plan. Also, the estimated taking yardage in the bid document should be a minimum of 30,000 cubic yards with an additive option of 20,000 cubic yards. A bid document similar to table D3 with the addition of an hourly rate should be adopted.

D.48 It should be noted that if the dredging boundaries are reduced and priorities are changed from those shown in figure D5, smaller quantities of material will be dredged. This reduces the trap size and, consequently, increases the number of phases required. It also reduces the length of time the harbor will remain open.

SECTION E

FEASIBILITY OF MITIGATING SHOALING EFFECTS

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(Continued)

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SECTION E

FEASIBILITY OF MITIGATING SHOALING EFFECTS

INTRODUCTION

E.1 Formulation and Evaluation Criteria. The primary objective in formulating and evaluating solutions was to keep the channel open to navigation at the least cost, while minimizing adverse effects and maximizing beneficial effects. In the littoral processes study estimated the rate of littoral transport was estimated and the shoaling mechanisms that had occurred in previous years under dredging practices in effect at the time were explained. Changing dredging procedures or altering the littoral regime by modifying the harbor protective works may substantially alter the quantity of material that would shoal in the channel. Therefore, each alternative was evaluated in light of how it could alter the shoaling processes and thereby affect the navigability of the channel.

E.2 The primary evaluation criterion was cost-effectiveness, which relates first costs and annual costs, including interest, amortization, maintenance, and operations costs, to the effectiveness of the system in maintaining the navigability of the channel. Consideration was also given to the needs of the commercial fishermen, who require a navigable channel by April 15 for salmon season. Effectiveness incorporates consideration of the number of days the harbor remains open to navigation, which is different for each alternative because the alternatives function on different principles.

E.3 Each alternative has potential environmental and social impacts. The primary impacts of each alternative on water quality, habitat, noise levels, and aesthetics were considered. The primary social considerations included obtaining easements, acceptability to the community, safety, and preservation of updrift and downdrift beaches. A secondary, but important, beneficial impact was the potential for improvement of the navigation channel either by providing additional protection to the entrance channel or by quieting the harbor basin.

E.4 In evaluating plans, consideration should be given to plan-effectiveness, which is the ability of a system to do a task as anticipated. All of the proposed alternative should be engineeringly feasible; however, certain risks that could render the program ineffective are inherent in each. Aside from the ability of the program to meet the primary objective of keeping the channel open, two basic determinants of its effectiveness are reliability and certainty. Reliability relates to the degree of success the system has experienced in the past. It assesses the susceptibility of the system to mechanical breakdown, strikes, delays in funding, etc. Certainty relates to the probability of success of a system assuming that the equipment is reliable. In evaluating certainty, it is assumed that the solution operates reliably, but consideration is given to the possibility that the littoral processes have not been defined accurately or that there is a large annual variation in littoral transport rates. An alternative whose effectiveness is independent of the littoral processes responsible for shoaling the harbor has a high certainty. One which is highly dependent on the presumed mechanism of shoaling has a low certainty.

E.5 <u>Alternatives</u>. Alternatives were classified into three categories: maintenance, bypassing, and structural. Maintenance pertains to the removal of material from the project channel and disposal of the material on the downdrift beach. Bypassing is a maintenance preventive, wherein sand is trapped or intercepted outside of the project channel and transported to the downdrift beach. A structural alternative either provides protection for a dredge or prevents material from entering the harbor. Structural solutions must be supplemented by some form of maintenance or bypassing. Several combinations of structural, maintenance, and bypassing systems are possible. The most effective combinations are presented and evaluated. Sixteen alternative solutions are listed in table E1. These alternatives were developed from analysis of the shoaling process and from evaluation of many possible solutions to the Santa Cruz problem.

E.6 <u>Disposal Sites</u>. Most of the alternative solutions have two features in common: (1) sand must be removed, and (2) it must be transported elsewhere. Descriptions of the alternatives include the methods of removal and quantities involved. A

TABLE E1

ALTERNATIVE SOLUTIONS

Maintenance

1. Annual Dredging - Floating Plant
2. Phased Dredging - Floating Plant
3. Hopper Dredge - "Currituck"
4. Mechanical Dredging Systems
5. Fixed Hydraulic System - Eductor
6. Fixed Hydraulic System - Zipper

Bypassing

7. Fixed Hydraulic System - Eductor
8. Mobile Hydraulic System - Eductor
9. Fixed Hydraulic System - Zipper
10. San Lorenzo River Sediment Trap

Structural

11. Long Offshore Breakwater - Annual Dredging
12. Short Offshore Breakwater - Continuous Bypassing
13. Extend West Jetty
14. Modify Both Jetties or Construct a New Entrance
15. Weir Jetty or Groin - Continuous Bypassing
16. Enhance Ebb Currents

common disposal site is used for most of the alternatives. In order to reduce erosion of downdrift beaches, the disposal site must be in the downdrift littoral zone, not in the offshore area. Black Point is a headland 3,000 feet downdrift of the harbor. Passing a discharge line around or over this headland would be major engineering task. Therefore, the primary disposal site is limited to the reach between the east jetty and Black Point. In past dredging episodes, material has been discharged approximately 1,000 feet downdrift of the east jetty. The interim dredging plan recommended disposal of the dredged material as close to Black Point as possible. The farther the sand is placed from the channel, the less it is apt to re-enter the channel. Two features prevented adequate discharge line extension during the first year of the two-year interim dredging contract. First, easements could not be obtained to extend the line more than 1,500 feet downdrift of the east jetty. Second, a series of severe storms concurrent with periods of high astronomical tides eroded the beach berm in December, 1977, eliminating the original discharge pipeline route.

E.7 The following disposal plan is recommended for most alternatives. The primary disposal site is 2,500 to 3,000 feet downdrift of the east jetty on the beach near Black Point as shown in figure E1. Disposal of material at this location increases the potential for waves to naturally bypass sand around the headland because the sand is at the downdrift end of the littoral cell. The discharge line should be buried approximately two to three feet below the beach for aesthetics and safety, as well as for protection of the pipe from unusually high wave runup. During the winter of 1977-1978, the beach was eroded to the base of a reveted bluff located about two-thirds of the distance to this disposal site. Therefore, the pipeline would either have to be buried in hard pan for about 500 feet along this bluff or be subjected to occasional wave runup, which could destroy it. Also, a discharge line must be rotated periodically, which is a difficult chore if the line is buried. The secondary disposal site is 1,000 feet east of the east jetty which would control beach erosion within the cell during severe winter storms and provide an alternative disposal site during repairs. Disposal of sand at the secondary site during the severe winter of 1977-1978 was required to protect the hinterland from erosion.

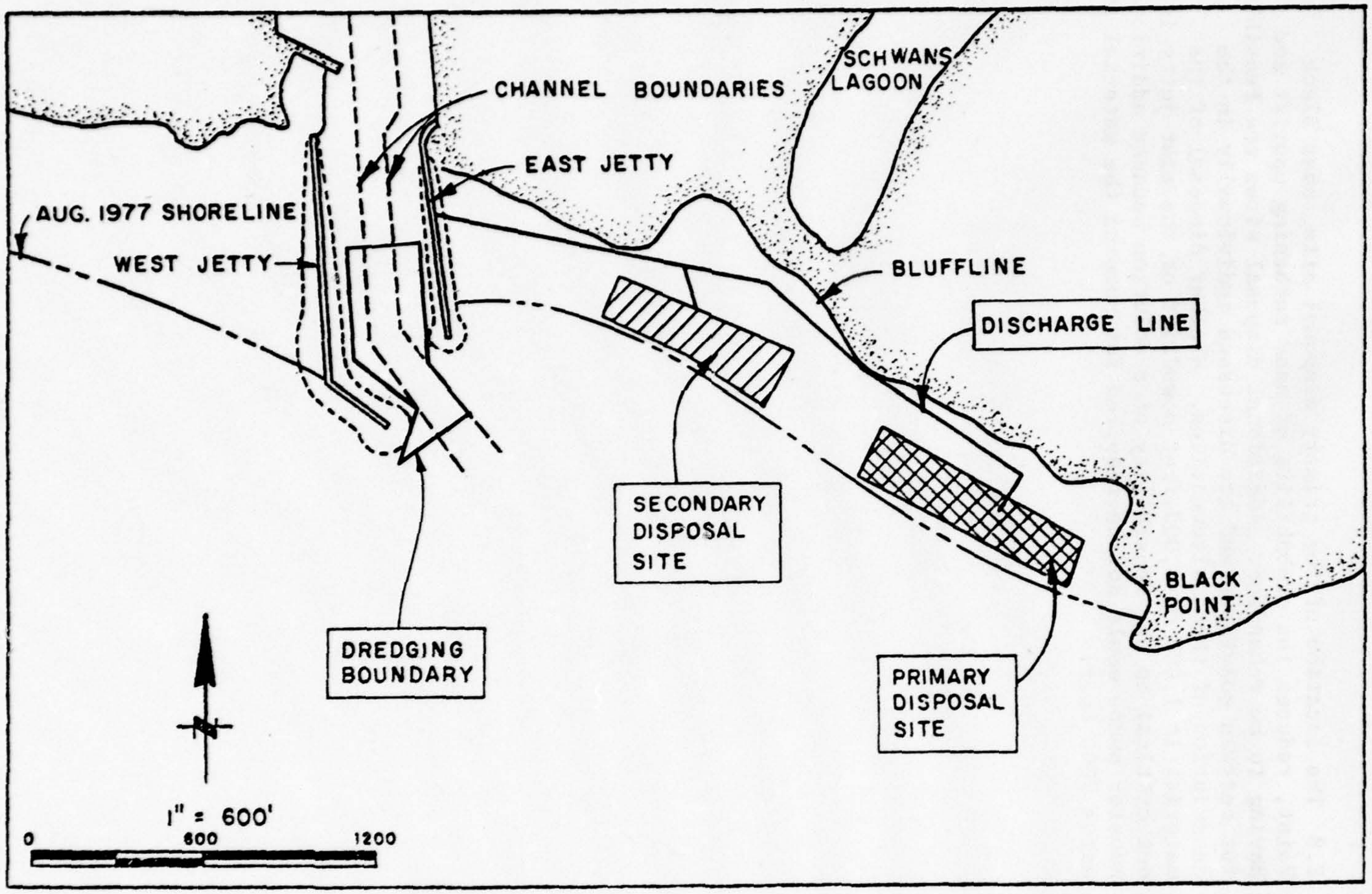

Figure E1. Disposal Sites and Dredging Boundaries for Alternatives 1 and 2.

E.8 The location of the primary disposal site, near Black Point, reduces the probability of sand returning updrift and having to be rehandled. Additional disposal sites are feasible for certain solutions and are discussed individually in the description of these alternatives. Whether disposal of the material is 1,000 or 3,000 feet downdrift of the east jetty is not critical to the feasibility of a solution because additional booster pumps would not be required for pumping the material an extra 2000 feet.

MAINTENANCE ALTERNATIVES

E.9 Alternative 1 - Annual Dredging - Floating Plant. Alternative 1 involves annual dredging of the entrance channel, which was the primary maintenance procedure at Santa Cruz until 1977-1978. The philosophy has been to mobilize a dredge in the late winter or early spring so as to remove the material from shoals in the channel. Periodic dredging is the typical maintenance procedure for many harbors around the world; however, this method has not been successful at Santa Cruz Harbor because of its small capacity for storing littoral drift.

E.10 The alternative comprises awarding a contract for dredging early in the calendar year, with work to begin in mid-to-late March. A 12- to 18-inch dredge is required, based on experience at Santa Cruz. Wave conditions during this period require that these dredges be equipped with adequate freeboard, extra-strength ladder, trunions, and swing gear, and that they have the ability to operate from wires and to excavate using a straight suction bell. The dredging boundaries are shown in figure E1. The acceptability of this system can be improved over past performances by optimizing procedures. The preferred procedure is to dredge a small pilot channel to a ten-foot depth initially to allow boat traffic to navigate the channel by April 15. Then, over-depth and over-width dredging within the confines of the jetties is recommended to increase storage capacity and thereby to extend the time period of channel navigability. Shoals or bars that accrete outside of the confines of the jetties should be removed as the last operation, when the waves are calmer.

E.11 Based on experience gained in previous dredging operations at Santa Cruz, a bid on the order of $150,000 for mobilization and $1.50 per cubic yard of excavation could be expected for this work. Previous "pay yardages" have been for 100,000 cubic yards, although 150,000 cubic yards have been estimated to shoal in the channel. Therefore, the bid at this price is assumed to reflect the bidding expertise of the contractor. No first costs are associated with this alternative; annual costs are given in table E2.

TABLE E2

COST ESTIMATE - Alternative #1

Floating Plant-Annual Dredging

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
FIRST COST - NONE				
ANNUAL COST				
Mobilization	1	LS		$ 150,000
Dredging (Pay Yards)	100,000	CY	1.50	$ 150,000
TOTAL ANNUAL COST				$ 300,000

E.12 In the past, periodic maintenance dredging has resulted in long periods of channel closure which has not been acceptable to boaters and fishermen. Under this alternative however, the floating pipeline in the channel would not interfere with navigation, since the dredge would be in the channel for a short time and only when the channel is already shoaled. The pipeline would be on the beach only during non-peak beach use in winter. Benthic organisms in the channel would be removed and organisms in the sand at the disposal site would be covered by the fill, but this has been ongoing for a number of years and is not known to have caused any major environmental problems or loss of endangered species. Disposal would not have a significant effect on kelp beds offshore because the dredging season would coincide with the period of low growth rates of kelp. This plan minimizes the quantity of material to be dredged because the shoal that forms across the mouth of the harbor in winter would not be disturbed and natural bypassing would not be reduced.

E.13 Alternative 2 - Phased Dredging - Floating Plant. The philosophy of phased dredging is to maintain the entrance channel for a greater length of time each year by removing the shoals periodically during the year rather than once annually. Section D describes the phased-dredging plan in detail. A phased dredging contract would be awarded late in the calendar year to mobilize a dredge onto the site by December. The dredge would be activated several times between December and April at times when the channel has shoaled to a minimum depth of ten feet. A final dredging episode in April would bring the channel to over-depth and over-width project dimensions to create as large a storage capacity as feasible with the present entrance configuration so as to maintain the channel until the following winter season. A 12-inch to 18-inch dredge would be used, because sea conditions during winter dredging would be more severe than during a one-time spring dredging plan. Cost estimates, based on removing 200,000 cubic yards per year, are given in table E3. Dredging boundaries are shown in figure E1. Care must be taken to verify the capability of the dredge to work in sea and swell conditions. A first cost item is also provided for purchase of a dredge to be permanently stationed and operated at Santa Cruz as a substitute for contract dredging.

TABLE E3

COST ESTIMATE - Alterntive #2

Floating Plant-Dredging Program

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
FIRST COST - NONE				
ANNUAL COST				
Mobilization (Dec. 15-Apr. 15)	1		LS	$ 150,000
Layup	2	Months	65,000	$ 130,000
Dredging	200,000	CY	1.50	$ 300,000
TOTAL ANNUAL COST				$ 580,000

NOTE:

A dredge could be purchased and operated by The Santa Cruz Harbor Port District. The resulting First and Annual Cost would then be $700,000 and $584,000 respectively.

E.14 The 1977-1978 interim dredging program used the phased dredging program and has revealed several problems, as discussed in Section D. The phased system proved more effective than annual dredging; however, dredging episodes could not be initiated as desired because of severe wave conditions that delayed pre-dredging surveys. Pre-dredging surveys could be eliminated by renting the dredge on an hourly basis or by having the Port District operate its own dredge. Alternative 2 has the advantage of high reliability and certainty. The dredging methods can be refined with experience to reduce costs and increase harbor usage.

E.15 Environmental impacts of Alternative 2 would be the same as those of the one-time dredging method described for Alternative 1. However, because the dredge would be in operation more often in periodic dredgings, the impacts would occur over a longer time span. The channel would be usable for a greater total length of time each year than it would under Alternative 1. The key to success with this system is to employ a fully competent contractor who has proved that he can work in rough sea conditions and to use an inspector who understands the shoaling process and sea conditions at Santa Cruz. Disposal of dredged material would be spread-out over the winter season. The phased dredging concept more closely approximates the natural littoral transport regime and would have a greater potential for stabilizing downdrift beaches and mitigating possible damages to them. A multi-year contract could reduce costs. If a new dredge were purchased, it could be used in conjunction with other projects to absorb some of the costs of debt retirement. Several years of multi-year contract experience should be obtained prior to considering the acquisition of a new dredge for the Port District to operate.

E.16 Alternative 3 - Hopper Dredge - The "Currituck". Many harbor entrances are maintained by hopper dredges. The principle of operation is the removal of material by a self-propelled suction dredge in open water. The hopper dredge has the capability of working in rougher seas than a pipeline dredge. It is possible that a small hopper dredge could maintain the channel at Santa Cruz and do minor bypassing from areas indicated in figure E2. Generally, hopper dredges discharge their hoppers in deep water; however, the Wilmington District of the U.S. Corps of Engineers has developed a split-hull hopper dredge, the "Currituck", that has the capability of depositing its dredged material in the nearshore zone.

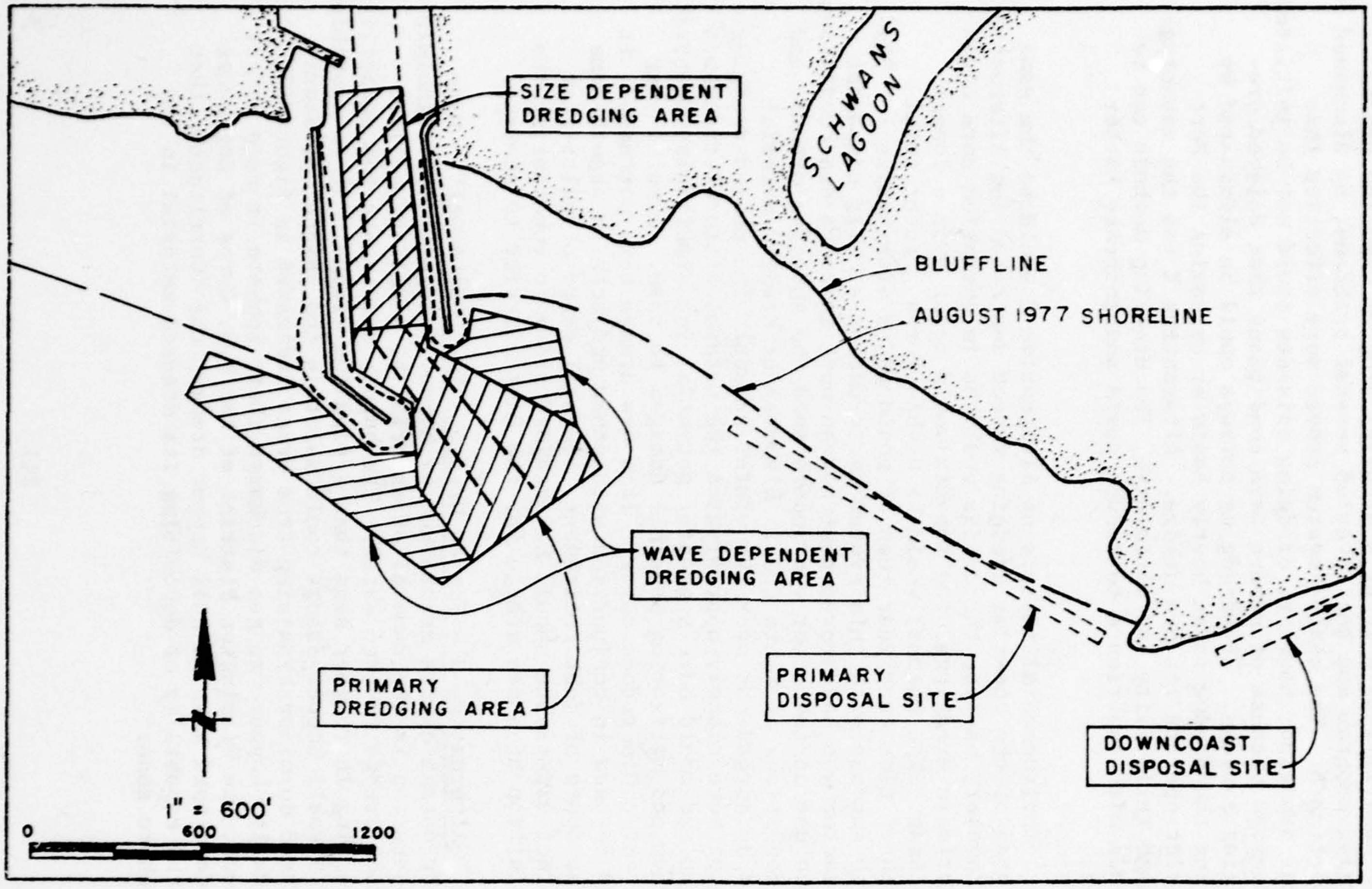

Figure E2. Disposal Sites and Dredging Areas for Alternative 3.

E.17 The agitation dredge works on another principle, and has been used successfully in special cases where fine material and moderate currents are found. The material is agitated by propeller wash, with or without a deflector, and currents are relied upon to remove the material. The characteristics of some recent hopper dredges and agitation dredges are given in table E4.

E.18 The two large conventional seagoing hopper dredges on the west coast whose characteristics are given in table E4, the "Pacific" and its planned replacement referred to as the "New Dredge", are too large to work Santa Cruz Harbor efficiently. The narrow channel severely restricts the maneuverability of a large dredge. Although the new dredge would be equipped with a bow thruster, allowing it to turn virtually within its own length, the only turning area available would be inside the jetties in the existing basin. The channel is only 100 feet wide, with less than 160 feet between the two jetties at the entrance. It would be extremely hazardous to maneuver either of the two large dredges in this confined opening on other than a calm day with no wind. The New Dredge is designed for use at a harbor on the Chetco River at Brookings, Oregon, where the jetties are about 200 feet apart and a 250-foot wide turning basin is provided at the inner end of the channel.

E.19 The dredge "Currituck", with its 140-foot length and 32-foot beam is probably the largest that can be used in the Santa Cruz Harbor channel. It would be restrained from working either in the dark or when combined sea and swell heights exceeded 4 feet. Operations would be limited in January and February; however, considerable work could be accomplished from October to December and from March to April. The "Currituck" is basically a split-hull barge equipped with twin engines, pumps, and drag arms designed for self-loading. However, it can be loaded by another dredge. The split-hull feature makes it possible to bottom-dump in shallow water by beaching the bow of the vessel, splitting the hull into the open position, and backing-off as the material is discharged. This allows the dredge to deposit material in shallow water near the beach where it can be moved by littoral forces as opposed to deep-water dumping.

E.20 The "Currituck" was constructed in 1975 at a cost of about $600,000. Its present rental rate is $1200 per day for 300 days of work per year. Production from the dredge is

TABLE E4

CHARACTERISTICS OF SOME RECENT HOPPER AND AGITATION DREDGES

	"SANDWICK"	"PACIFIC"	"NEW DREDGE"	"CURRITUCK"
Type	Agitation	Hopper	Hopper	Hopper
Length (ft)	74	185	200	140
Beam (ft)	21	38	45	32
Draft (ft)	5	9	8	4
Light	-	11.5	12	8
Hopper Capacity (cy)	-	500	800	300

TABLE E5

PRODUCTION RATES OF THE "CURRITUCK"

		Cubic Yards
November:	95% x 4 weeks x 10,000 =	38,000
December:	85% x 4 weeks x 10,000 =	34,000
January:	80% x 4 weeks x 10,000 =	32,000
February:	55% x 4 weeks x 10,000 =	22,000
March:	80% x 4 weeks x 10,000 =	32,000
April:	85% x 4 weeks x 10,000 =	34,000
Total During Winter		192,000

approximately 2000 cubic yards per day, or 10,000 cubic yards per week based on a 10-hour work day and a 5-day week. Rough weather would necessitate suspension of its operation. Assuming the dredge would only work when combined sea and swell height are less than 4 feet, work times and production would be as given in table E5. The pumping rate is less than the 4000 cubic yards per day specified for the interim dredging program.

E.21 Assuming that the "Currituck" could be used on other projects during the year at other locations and that the rental rate is $36,000 per month, dredging costs would be $1.20 per cubic yard. Cost estimates based on dredging 200,000 cubic yards a year and a 10-year life are given in table E6.

E.22 This system has been undergoing tests on the east coast in the Atlantic by the Corps of Engineers. Although test results have been encouraging, the question remains as to whether the system would be effective at Santa Cruz. The channel is narrow and the wave climate is rough, with high waves and winds occurring during the periods when the dredge is needed the most. A major disadvantage of this sytem is the possibility of a severe winter storm closing the harbor thereby landlocking the dredge, which requires a seven-foot depth for operation. A pump-out system perhaps could be devised; however, this would increase costs and its feasability has not been explored.

E.23 The system has a distinct advantage of being capable of bypassing material farther downdrift past Black Point if required, without the necessity of placing an unsightly pipeline on the beach. However, nearshore wave action may force the dredge to deposit the material farther offshore than desired. This sytem has potential, but several unknowns and possible modes of failure suggest a low reliability.

E.24 The environmental impacts of the hopper-dredging alternative are modest. Material would be placed in the littoral zone which would cause temporary turbidity and discoloration of the water and cover benthic organisms. However, this is a zone where continual sand movement is taking place under natural conditions. The dredging equipment would be in the channel

TABLE E6

COST ESTIMATE - Alternative #3

Hopper Dredge

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
FIRST COST				
Construction Cost-Based on Cost of "Curri-tuck"			LS	$1,500,000
Contingencies (15%)				$ 225,000
Subtotal				$1,725,000
Engineering & Design (10%)				$ 172,000
Supervision & Admin. (7%)				$ 121,000
TOTAL FIRST COST				$2,018,000
ANNUAL COST				
Personnel (Nov.-Apr.)				$ 118,000
Fuel & Oil				$ 15,000
Maintenance				$ 75,000
Support Personnel & Misc.				$ 50,000
Interest (6-5/8%)				$ 134,000
Amortization (20 Yr.)				$ 52,000
TOTAL ANNUAL COST				$ 444,000

much of the winter, but the dredge does not connect to a pipeline and has maneuverability to avoid other traffic using the channel.

E.25 The other sea-going dredge selected for analysis is the "Sandwick", owned and operated by the Portland District of the Corps of Engineers. This vessel is a converted Navy LCM which has a deflector mounted on the stern. The vessel is anchored in place and the deflector is lowered. The throttles are then opened and the resulting propeller wash is directed towards the shoal. This agitation dredging method is particularly effective with fine sand and silty material when strong river or tidal currents are present. The dredge is capable of displacing 200 to 500 cubic yards per hour and moving it several hundred feet at costs generally under $1.00 per yard. However, Santa Cruz does not have fine material or an adequate current to carry the dislodged material out of the channel. Therefore, agitation dredging was not considered to be engineeringly feasible for this project.

E.26 Environmental impacts of using the "Sandwick" primarily relate to the turbidity caused by the dredge when it dislodges the material. However, because the material is sand with few fines, this impact should be neither long lasting nor serious. Anchoring the dredge in the channel would cause some hazard to navigation, but if the anchors were judiciously placed, this should not be any greater problem than that created by pipeline dredging.

E.27 <u>Alternative 4 - Mechanical - Shore-Based Dragline/Crane</u>. Mechanical systems comprise operating a dragline, a crane with a clamshell bucket, or a Sauerman system from either or both jetties. Excavation by dragline or crane is a common earth-moving procedure that need not be described in detail. In principle, a Sauerman bottomless bucket is pulled back and forth over the borrow area by a system of winches, fairleads and sheaves on a head tower or crane on the near side and on a deadman, such as a tail tower, anchor barge, dolphin system on the far side. When pulled, the bucket excavates its load and drags it to the stockpile area. When dragged back to the far side, only the bucket moves, leaving its last load on the stockpile. The direction can easily be reversed to move the material to the far side by simply reversing the attachments to the bucket. The Sauerman system extends the reach of the dragline or crane.

E.28 The Sauerman principle could be applied at Santa Cruz in several ways; two ways are shown in figure E3. A pair of dolphins or an anchored barge would have to be stationed offshore to act as a deadman. Assuming an average pull length of 800 feet with a 10-cubic yard bucket operating ten hours per day at 40-percent bucket efficiency in mild waves, the rated daily capacity is 640 cubic yards. This system has been used successfully in slack or protected water; however, the system is essentially unproven for work in the surf zone. The impact of the wave energy and long-shore currents on the bucket would be severe and could make it impossible to dredge except during low-wave episodes. Even if the system were feasible and could be proven to operate at Santa Cruz, the material would have to be rehandled at the inshore end. A fixed or floating dredge, or truck haul, would be required to transport the material across the channel and down the beach. Reversing the attachments for disposal on the offshore stroke could pose a potential navigation problem by creating a dangerous shoal. Because of the unproven nature of the equipment, the fact that the material would have to be rehandled, and the hazards to navigation, it was concluded that this system did not have sufficient viability to warrant making detailed cost estimates.

E.29 Another mechanical dredging system analyzed was a crane-mounted clamshell or a dragline working from the head of the east or west jetties, or from both, as shown in figure E4. While equipment is manufactured that could swing a dragline bucket far enough to cover nearly all shoal areas in the project channel, this equipment is very specialized and probably would not be available for working intermittently during the year. An alternative solution would be to build a trestle extending from the jetty which would allow a smaller dragline to reach the channel. Another alternative would be to mount the dragline on the east jetty and have a truck-mounted, single-drum winch on the west jetty. The truck-mounted winch would manipulate a tail line to haul the dragline bucket beyond the radius of the boom. In this manner nearly all of the channel areas subject to severe shoaling could be reached by the dragline bucket. The largest feasible capacity for this type of operation, assuming a 4 cubic yard, 60-percent-full bucket, and a one-minute digging cycle, would result in production of 150 cubic yards per hours. The production rate for a 10-hour day, with 20 percent down-time, would be 1,2000 cubic yards per day. Cost estimates are given in table E7.

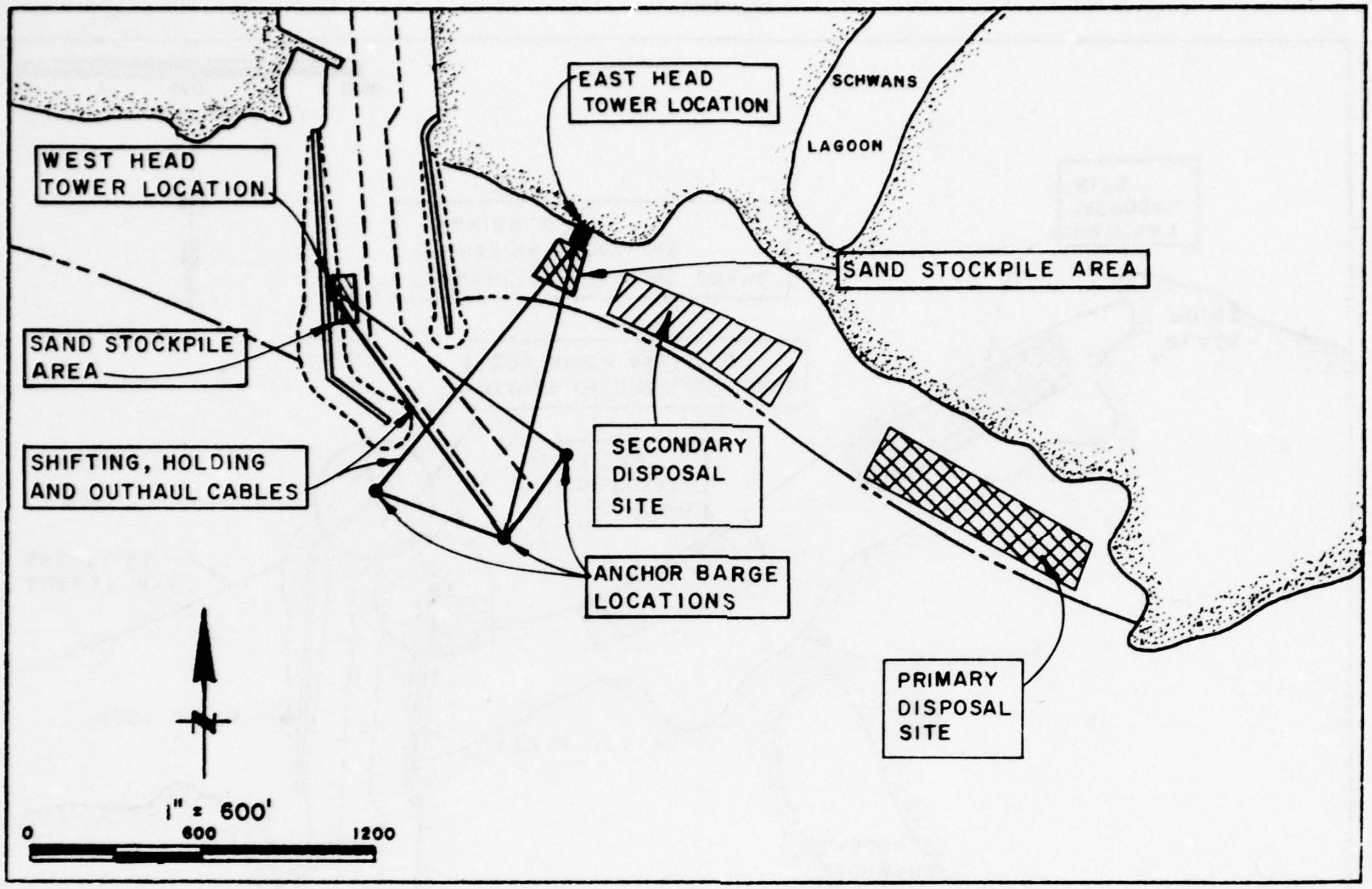

Figure E3. Mechanical System - Alternative #4 - Sauerman Principle

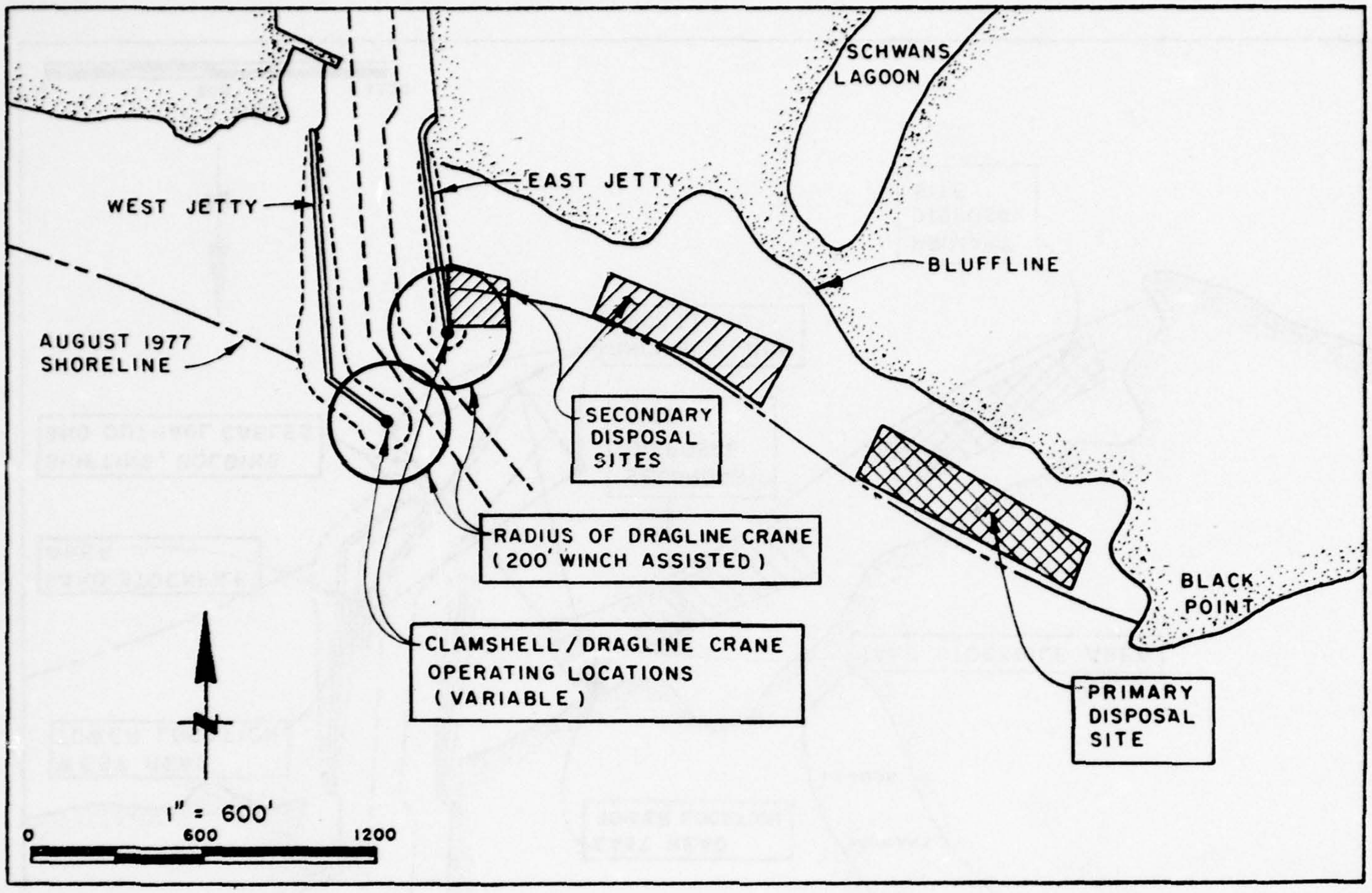

Figure E4. Mechanical System - Alternative 4 - Crane-mounted Clamshell/Dragline

TABLE E7

COST ESTIMATE - Alternative #4

Dragline/Crane Operation

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
FIRST COST - NONE				
ANNUAL COST				
Rental Cost				
Dragline Crane 4 CY (Rental includes operator)	10	Hr.	150.00	$ 1,500
10 CY. Trucks (3)(Rental includes driver)	30	Hr.	35.00	$ 1,050
Truck Mounted Winch (Rental includes operator)	10	Hr.	30.00	$ 300
Foreman or Supervisor	10	Hr.	15.00	$ 150
Oiler	10	Hr.	13.00	$ 130
DAILY OPERATING COST (10 Hr. day)				$ 3,130
Cost per CY.	$3,130/Day/1200 CY./Day = $ 2.61/CY.			
Dredging Cost	200,000 CY. x $2.61/CY. = $522,000			
TOTAL ANNUAL COST				$ 522, 000

E.30 The dragline and cables would be working essentially across the channel. This would hinder navigation. The line could be dropped to allow passage of individual boats; however, the already low production rates would decrease. Trucking material along the beach to the disposal area would be both unsightly and dangerous. The area along the east jetty and along the beach would have the appearance of a construction project for the entire time that work was underway. This would be incompatible with the level of recreational use during good-weather periods.

E.31 The system has several other drawbacks. Production of 1,200 cubic yards per day would probably not keep pace with the rapid shoaling during storms. A long period of time would elapse after a storm before the channel could be restored to project dimensions. This system has a low degree of reliability working in breaking waves. The lateral force of the breaking waves against the bucket could result either in partial bucket loads or a cessation of work.

E.32 Alternative 5 - Fixed Hydraulic System - Eductor. An eductor or jet pump system is a recent development in sand bypassing methods. Clear, high-pressure water is pumped to a nozzle which converts it into a high-velocity, low-pressure jet stream. The suction created by the partial vacuum induced by the jet entrains sand, which is mixed with the water jet and discharged through a pipeline. The sand and water mixture is then pumped to the downdrift beach, aided by a booster pump. The basic principle of operation has been to lower an eductor into the sand and allow the eductor to excavate a crater. Wave action and currents theoretically feed the crater. This system has been used experimentally at Mexico Beach, Florida, Rudee Inlet, Virginia, and at Santa Cruz by WES as described in Section C.

E.33 The experimental system was not designed to maintain, nor has it maintained, the Santa Cruz channel. Figure E5 shows a possible application of the eductor system at Santa Cruz designed to maintain the channel. The principle is to install ten eductors at 60-foot spacings in the project channel at a 37-foot depth. Operation of one eductor at a time would create a line of overlapping craters to maintain project dimensions. A pump house

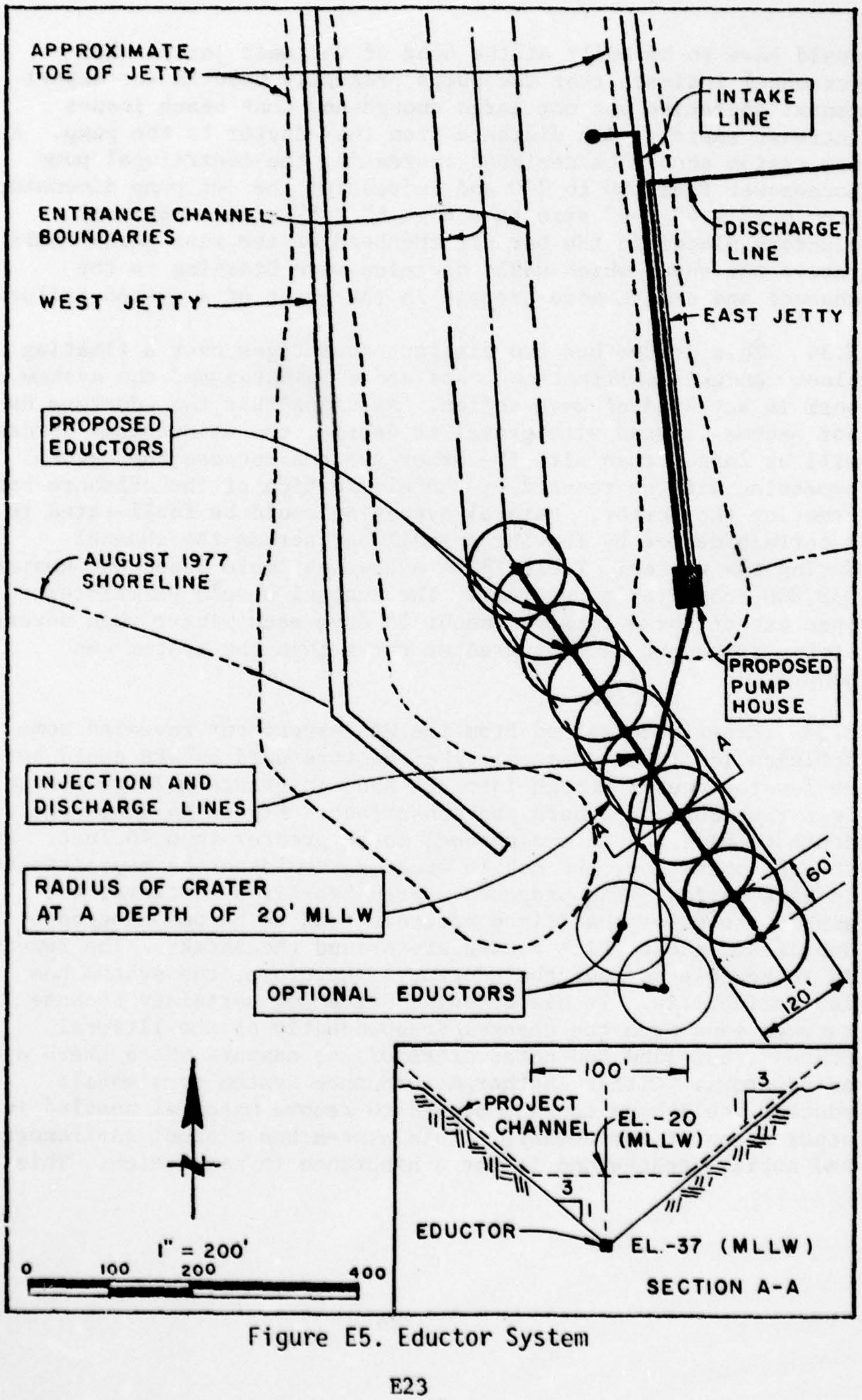

Figure E5. Eductor System

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would have to be built at the head of the east jetty. WES personnel indicate that the pumps presently used in the experimental operation are not large enough and that beach losses increase rapidly with distance from the eductor to the pump. A new system should be designed increasing the centrifugal pump horsepower from 300 to 700 and increasing the jet pump dimensions from a 4" x 4" x 6" size to a 6" x 6" x 8" size. Optional eductors placed in the bar off the head of the west jetty could remove the shoal, which would decrease wave breaking in the channel and create more storage in the event of a system failure.

E.34 This system has two distinct advantages over a floating plant. Annual mobilization costs are eliminated and the system can work in any kind of wave action. Assuming that the eductors do not become clogged with gravel or debris, the maintenance yardages will be larger than with the other systems because the natural bypassing will be reduced by the elimination of the offshore bar fronting the harbor. Natural bypassing could be facilitated to a certain degree by leaving a small bar across the channel during the winter. Table E8 is a cost estimate based on pumping 250,000 cubic yards per year. The channel should be maintained open except for a total of about 15 days each winter when severe storms transport sand at greater rates than the system can handle.

E.35 Experience gained from the WES experiment revealed some deficiencies in the system. The eductors used by WES could not be lowered deeply enough into the sand to create a large enough crater because of a hard pan subsurface. Figure E6 shows the depth to hard pan in the channel to be greater than 40 feet. In the WES operation, the mobile eductors could not be repositioned in wave action. The proposed system has fixed eductors, but experience shows that fixed eductors tend to become clogged by debris and rocks which accumulate around the intake. The remedy is to be able to move the eductor. Therefore, the system has low reliability. It has a medium degree of certainty because it removes sand from the channel independently of the littoral process, but sand can enter areas of the channel where there are no eductors. Either another maintenance system or a mobile eductor would have to be employed to remove material shoaled in other areas of the channel. This system has minimal environmental and social impacts and is not a hindrance to navigation. This

TABLE E8

COST ESTIMATE - Alternative #5

Eductor-Channel Maintenance

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
FIRST COST				
Pumphouse				
Cap Stone Removal				$ 9,200
Bed Grout				$ 50,750
Concrete				$ 51,500
Roof				$ 20,000
Subtotal				$ 131,500
Clear Water Intake				
Intake Structure				$ 15,000
Buried Pipe				$ 4,500
Riser Section				$ 4,000
Top of BW Run				$ 38,500
Subtotal				$ 62,000
Pumping Equipment				$ 100,000
Jet Pumps	10	EA	2,000	$ 20,000
Installation				$ 25,000
Misc. Piping				$ 50,000
Connect Power				$ 45,000
Subtotal				$ 240,000

TABLE E8 (Continued)

Item	Quantity	Unit	Unit Price	Cost Estimate
Injection & Return Lines	7,000	LF	50	$ 350,000
Trenching & Connections				$ 100,000
Shore Discharge Line	2,500	LF	50	$ 125,000
Trenching & Anchors				$ 50,000
Subtotal				$ 625,000
Construction Contingencies (25%)				$ 264,000
Subtotal				$1,322,000
Engineering & Design (10%)				$ 132,000
Supervision & Admin. (7%)				$ 92,000
TOTAL FIRST COST				$1,546,000
ANNUAL COST				
Power (250,000 CY/400 CY/HR) X 1575 HP X $0.055/KW-HR				$ 54,000
Maintenance				$ 50,000
Pipe Turning				$ 15,000
Personnel				$ 50,000
Interest (6-5/8%)				$ 102,400
Amortization (10 yrs.)				$ 114,000
TOTAL ANNUAL COST				$ 385,000

NOTE:

Optional Eductors would add $135,000 to the First Cost and $19,000 to the Annual Cost.

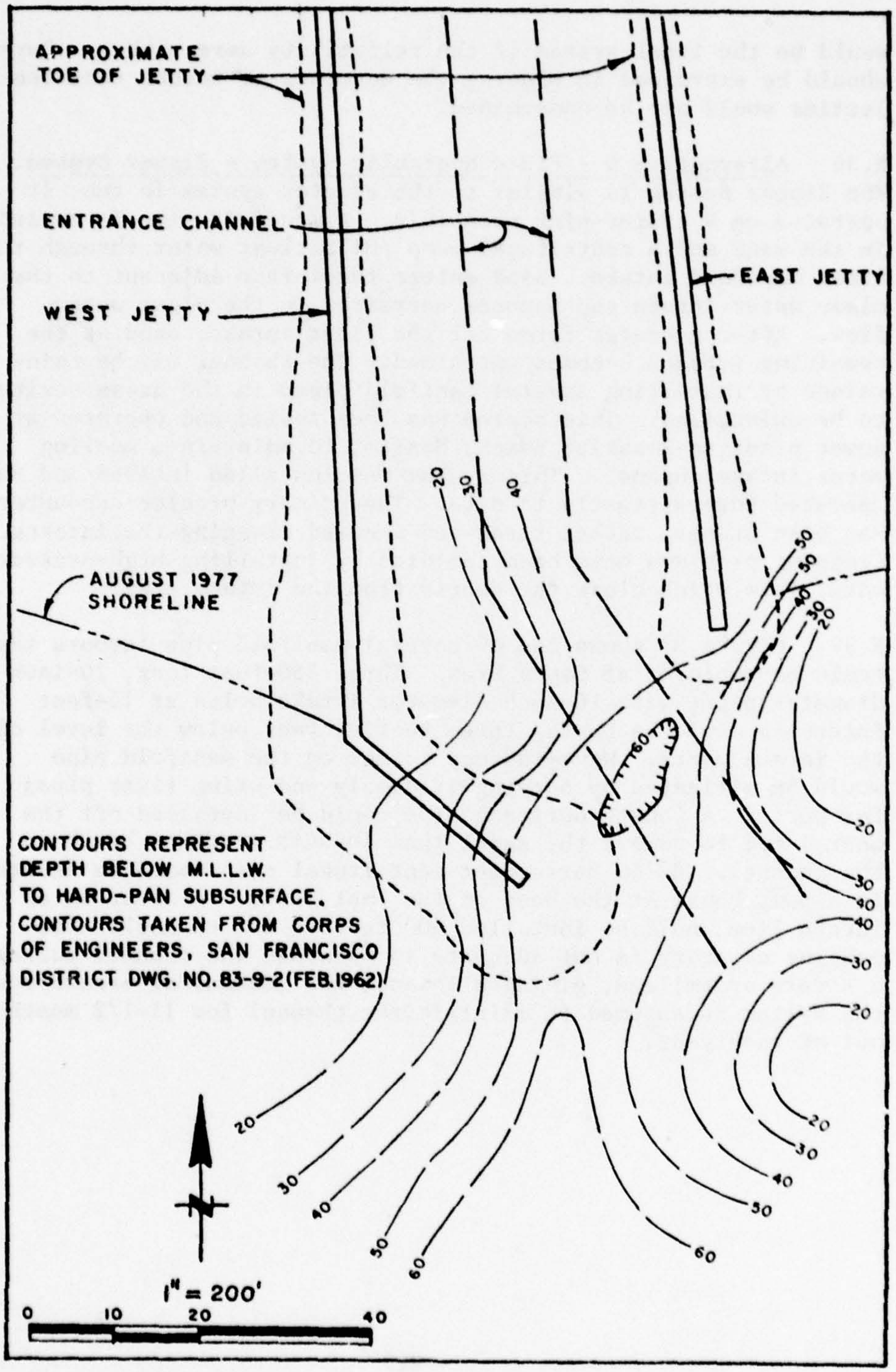

Figure E6. Depths to Hardpan

would be the ideal system if the reliability were higher. Care should be exercised in placing the eductors to ensure that the jetties would not be undermined.

E.36 Alternative 6 - Fixed Hydraulic System - Zipper System. The Zipper System is similar to the eductor system in that it operates on a crater-sink principle. A manifold pipe is buried in the sand and a centrifugal pump pulls clear water through the first manifold intake. Sand enters the intake adjacent to the clear water intake and becomes entrained in the clear water flow. After a crater forms off the first intake, sand at the remaining intakes becomes entrained. The channel may be maintained by installing several manifold pipes in the areas desired to be maintained. This system has been tested and operated at a power plant in Rosarito Beach, Mexico, to maintain a cooling water intake channel. This system was installed in 1968 and has operated intermittently to date. The primary problem encountered has been stones, rubber tires and seaweed clogging the intakes. Clogging problems have been remedied by installing high-pressure water jets which clear the debris from the intake area.

E.37 Figure E7 shows one of several manifold pipe layouts that could be employed at Santa Cruz. Three 150-foot long, 20-inch diameter pipes with 10-inch diameter intake holes at 15-foot intervals would be buried three to five feet below the level of the intake ports. Wave-induced forces on the manifold pipe would be mitigated by burying it deeply and using riser pipes for ports. A fourth optional pipe could be installed off the west jetty to remove the shoal that induces waves to break in the channel. A 750-horsepower centrifugal pump would be located in a pump house at the head of the east jetty. A clear water intake line would be installed 500 feet up the channel. The pumping capacity is not adequate to maintain the channel during a severe storm; and, giving allowance for mechanical breakdowns, the system is assumed to maintain the channel for 11-1/2 months out of each year.

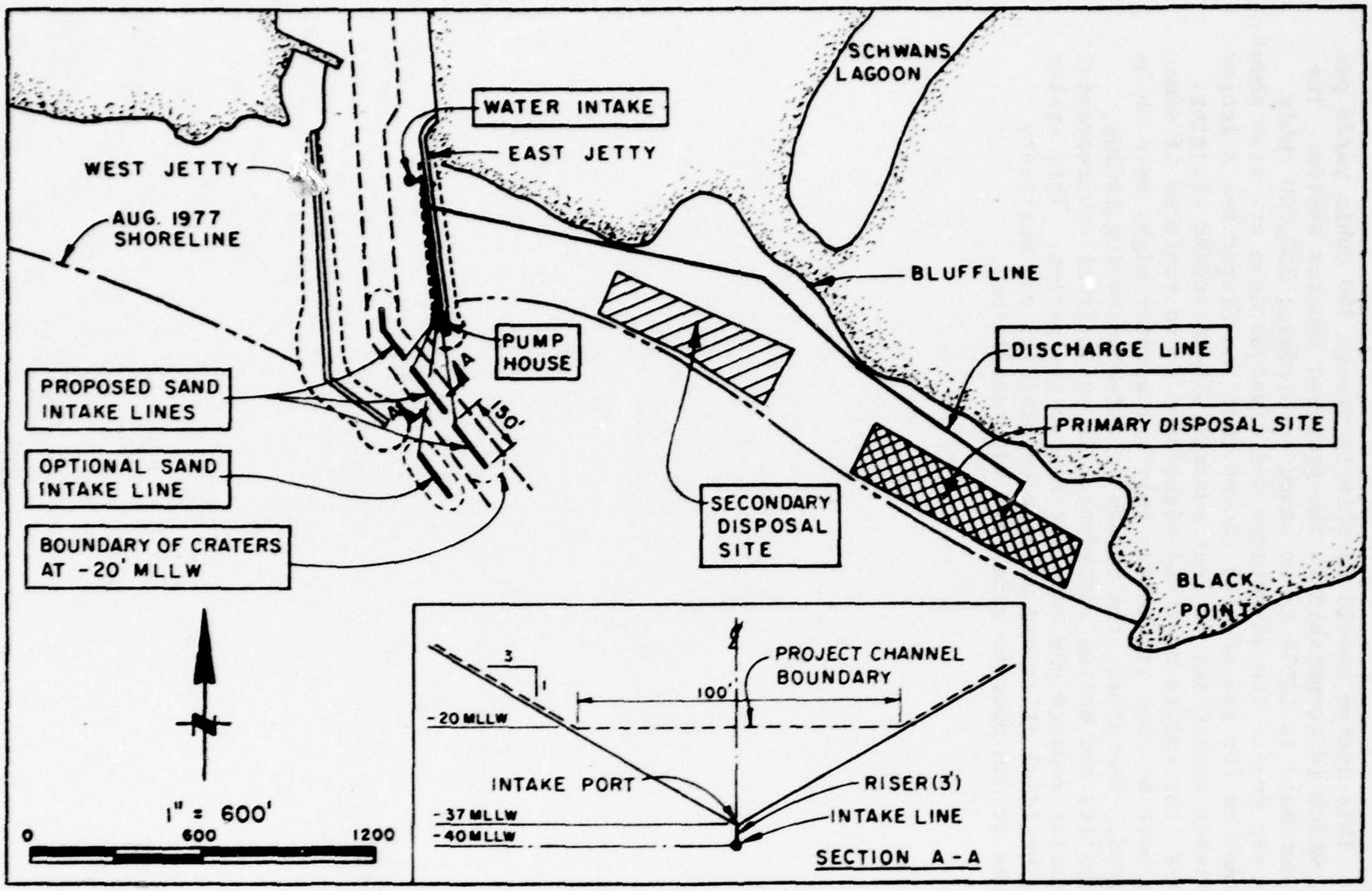

Figure E7. Zipper

E.38 This system should be able to dredge 350 cubic yards per hour, which is comparable to the proposed eductor system. The cost estimate in table E9 is based on dredging 250,000 cubic yards per year. The advantages and disadvantages are also about the same as for the eductor except that the Zipper has a longer experience record and is less susceptible to intake clogging. However, the entire manifold might have to be replaced if something were to clog it whereas only one eductor might have to be replaced. Therefore, the system is rated as having medium reliability and medium certainty. It has minimal environmental and social impacts and does not hinder navigation. This system would be ideal if it had greater reliability and certainty because it can operate under all wave conditions.

TABLE E9

COST ESTIMATE - Alternative #6

Zipper Channel Maintenance

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
FIRST COST				
Pumphouse				
Cap Stone Removal				$ 9,200
Bed Grout				$ 50,750
Concrete				$ 51,500
Roof				$ 20,000
Subtotal				$ 131,500
Sand Intake Lines (3)				
Manifold Line (20"Ø)	450	LF	200	$ 90,000
Jet System	450	LF	125	$ 56,250
Slide Valve	3	EA	5,000	$ 15,000
Pipe Runs to Pump	3	EA	7,000	$ 21,000
Place Pipe Lines	660	LF	350	$ 231,000
Subtotal				$ 413,250
Water Intake Line				$ 62,000

TABLE E9 (Continued)

Item	Quantity	Unit	Unit Price	Cost Estimate
Pumping Equipment				
Pump & Motors			LS	$ 50,000
Connect Power				$ 45,000
Controls				$ 100,000
Misc. Piping				$ 60,000
Subtotal				$ 255,000
Discharge Line	2,500	LF	50	$ 125,000
Trenching & Anchors		LS		$ 50,000
Subtotal				$ 175,000
Construction Contingencies (25%)				$ 259,000
Subtotal				$1,296,000
Engineering & Design (10%)				$ 130,000
Supervision & Admin. (7%)				$ 91,000
TOTAL FIRST COST				$1,517,000
ANNUAL COST				
Power (250,000 CY/350 CY/HR) X 750 HP x $0.055/KW-HR				$ 30,000
Maintenance				$ 50,000
Pipe Turning				$ 15,000
Personnel				$ 50,000
Interest (6-5/8%)				$ 100,000

TABLE E9 (Continued)

Item	Quantity	Unit	Unit Price	Cost Estimate
Amortization (10 yrs.)				$ 112,000
Royalty	250,000	CY	$0.20	$ 50,000
TOTAL ANNUAL COST				$ 407,000

NOTE:

Optional Sand Intake Line will add $229,000 to the First Cost and $32,000 to the Annual Cost.

BYPASSING SYSTEMS

E.39 General. The littoral processes study indicated that most of the sand enters the chanel by passing around the head of the west jetty or by leaking through its voids. A typical solution is to install a fixed bypassing plant on the updrift beach to bypass material that accumulates in a fillet impounded by the updrift jetty. This was the apparent intent of the Authorizing Document. Three schemes presented below are based on installing a permanent plant to operate from the west jetty. The bypassing plants must have capability to pump sand at a great enough rate to preclude sand from bypassing the jetty and, consequently, shoaling the channel. The WES eductor experiment bypassed sand in this manner in 1977. Prior to the severe winter, its result was a decrease of sand entering the control area used for volumetric calculations. The first storm, however, erased any apparent effects of this effort. The separate effects of the eductor and dredge systems could not be assessed properly for lack of data sufficiently detailed for such an analysis and because of the severity of the winter.

E.40 Alternative 7 - Fixed Hydraulic System - Eductor. The major disadvantage of existing fixed bypassing plants is their inability to reach all material in littoral transport. The plants generally have booms with reaches less than 100 feet. These plants are usually located on short jetties, and sand readily bypasses them via offshore bars and ripcurrents. Several techniques including installation of fluidizing lines or draglines to extend the reach of the system may be promising, but they have not been proven in the field. Figure E8 shows a proposed bypassing system with five eductors located on the updrift side of the west jetty to create a sand trap. The west beach could be controlled by pumping from eductors progressively from the offshore unit to the inshore unit. Waves generating longshore currents would transport material to the trap. Eventually the beach would recede to the 1965 position. Severe storms would fill the fillet, but only a fraction of the volume that presently moves around the head of the west jetty would still bypass that structure. The cost estimate given in table E10, is based on using the same basic eductor system as proposed for the eductor maintenance program. A major difference

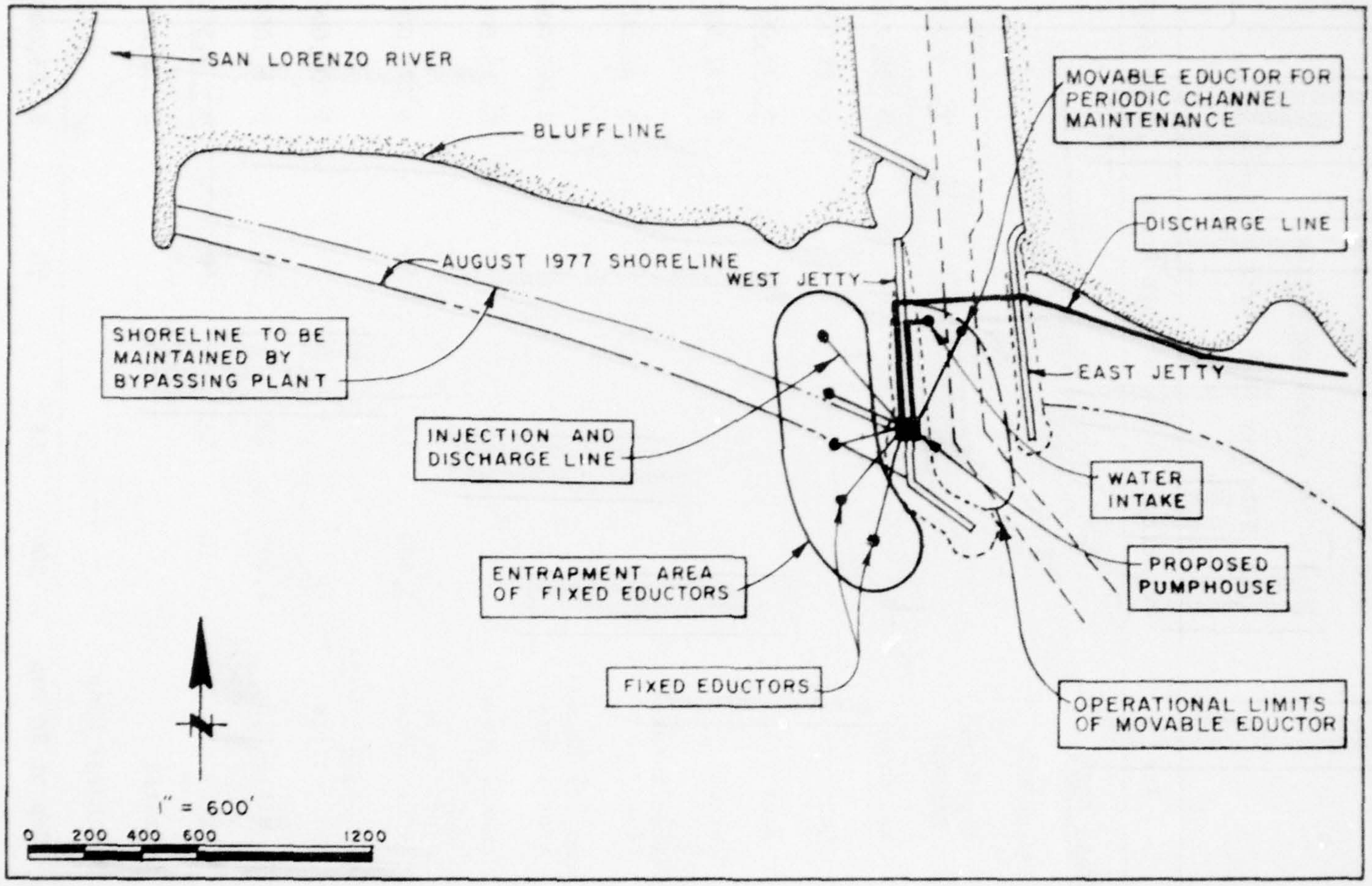

Figure E8. Fixed Eductor Bypassing

TABLE E10

COST ESTIMATE - Alternative #7

Eductor Bypassing System

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
FIRST COST				
Pumphouse				
Cap Stone Removal				$ 9,200
Bed Grout				$ 50,750
Concrete				$ 51,500
Roof				$ 20,000
Subtotal				$ 131,500
Water Intake Line				$ 66,000
Pumping Equipment & Jets				$ 195,000
Connect Power				$ 45,000
Injection & Return Lines	3,400	LF	50	$ 170,000
Trenching & Misc. Connections				$ 100,000
Flexible Line	1,000	LF	50	$ 50,000
Floats	10	EA	250	$ 2,500
Subtotal				$ 322,000
Discharge Line				
Top of BW Run	500	LF	50	$ 25,000
Anchors @ 10'	50	EA	350	$ 17,500

TABLE E10 (Continued)

Item	Quantity	Unit	Unit Price	Cost Estimate
Channel, Crossing				
Riser Sections	2	EA	4,000	$ 8,000
Pipe 10"Ø-Buy & Place & Bury	350	LF	90	$ 31,500
Shore Run				
10"Ø-Buy & Place & Trench	2,000	LF	50	$ 100,000
Subtotal				$ 177,000
Construction Contingency (25%)				$ 234,000
Engineering & Design (10%)				$ 117,000
Supervision & Admin. (7%)				$ 82,000
TOTAL FIRST COST				$1,370,000
ANNUAL COST				
Power (300,000 CY/400 CY/HR) X 1575 HP X $0.055 KW-HR				$ 65,000
Maintenance				$ 50,000
Pipe Turning				$ 20,000
Personnel				$ 50,000
Interest (6-5/8%)				$ 91,000
Amortization (10 yrs.)				$ 101,000
TOTAL ANNUAL COST				$ 377,000

between the bypassing system and the maintenance system is that five eductors are used in the bypassing program versus the ten in the maintenance program. Some sand would bypass around the head of the jetty, and shoaling due to other littoral processes, such as updrift and onshore transport, would continue. A portable eductor for removal of shoals in the channel is included in the cost estimate. This eductor would be positioned by a small boat during periods of low-wave action.

E.41 This system is rated low in reliability because it is not proven. Sand can enter the channel from sources other than the eductor area, and the eductor craters provide insufficient storage to preclude shoaling during a severe storm. The certainty is low because the solution relies heavily on the hypothesis that channel shoaling results only from sand bypassing the west jetty or sand leaking through the voids in it. Environmental and social effects of dredging the west beach back approximately 300 feet would be objectional to many persons.

E.42 Alternative 8 - Mobile Hydraulic System - Eductor. The reach of a fixed bypassing plant could be extended by using a track-mounted mobile dredge on a trestle, or a truck-mounted crane on a jetty. Figure E9 shows an envelope that a series of craters could create by suspending an eductor from a 100-foot long boom on a truck-mounted crane. The truck would operate on the cap of the west jetty. Power and pumps would be located on a trailer attached to the mobile crane. The discharge line and clear water intake line would be permanently attached to a fixed manifold which would be located on the west jetty. The system would operate primarily on the updrift beach as a bypass system. However, by swinging the boom to the channel side of the jetty, the shoulder of the channel could be dredged and maintained. The salient fronting the west jetty dogleg section around the head could also be dredged during calm wave conditions. Maintenance dredging in the navigation channel to maintain the project dimensions could be performed by a float-mounted eductor using a floating extension of the suction line. Cost estimates given in table E11 are based on the assumption that 250,000 yards of material would be bypassed and 50,000 yards of material would be removed by maintenance dredging.

E.43 The system is not reliable in adverse weather conditions. High winds and overtopping waves would keep the crane from

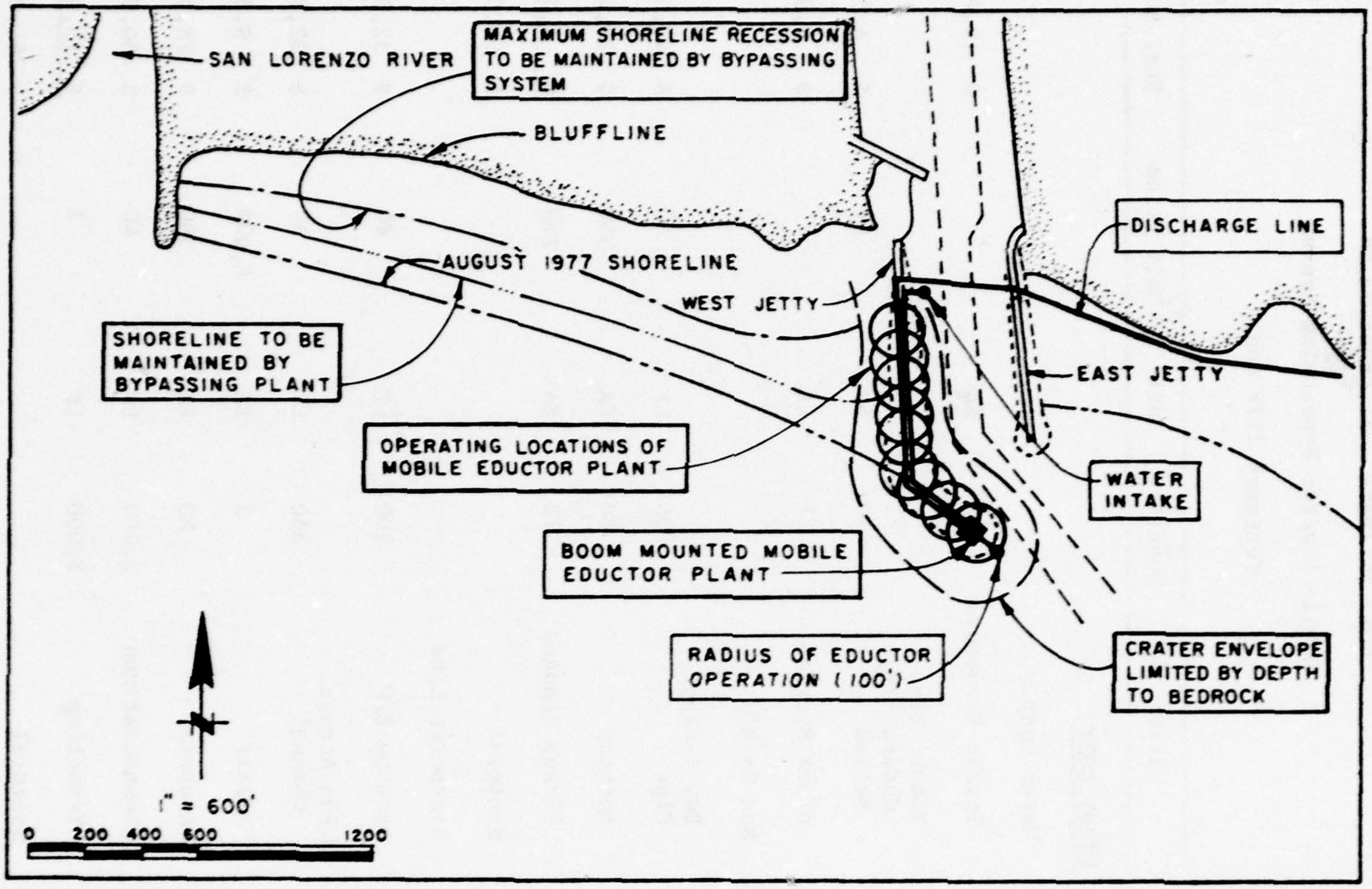

Figure E9. Mobile Eductor

TABLE E11

COST ESTIMATE - Alternative #8

Mobile Eductor Bypassing System

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
FIRST COST				
Water Supply				
Intake Structure	1	EA		$ 15,000
Intake Line Underwater, Buried	50	LF	90	$ 4,500
Riser Section	1	EA		$ 4,000
Run On B/W				
Buy & Place Pipe	800	LF	50	$ 40,000
Anchors	80	EA	350	$ 28,000
Pickup Flanges	18	EA	250	$ 4,500
Subtotal				$ 96,000
Discharge Line				
Run On B/W	800	LF	40	$ 32,000
Run Across Channel	350	LF	90	$ 32,000
Risers	2	EA	4,000	$ 8,000
Anchors	80	EA	350	$ 28,000
Downcoast Run	2,000	LF	40	$ 80,000
Trenching	2,000	LF	7	$ 14,000
Subtotal				$ 194,000

TABLE E11 (Continued)

Item	Quantity	Unit	Unit Price	Cost Estimate
Mobile Dredge/ Pickup Diesel Powered Crane 80 T Lima		EA		$ 190,000
Revamp				$ 80,000
Pumps & Eductor				
Buy & Install. Allow				$ 80,000
Pipeline On Boom	260	LF		$ 15,000
Trailer Mount & Fuel Tank				$ 75,000
Floating Extension				
Flexible Line	500	LF	50	$ 25,000
Floats	8	EA	250	$ 2,000
Subtotal				$ 467,000
Contingencies (25%)				$ 189,000
Subtotal				$ 946,000
Engineering & Design (10%)				$ 95,000
Supervision & Admin. (7%)				$ 66,000
TOTAL FIRST COST				$1,107,000

TABLE E11 (Continued)

Item	Quantity	Unit	Unit Price	Cost Estimate
ANNUAL COST				
Fuel-				
Bypassing				
(250,000 CY/200 CY/HR) X 1000 HP X 1/8 GAL/HP-HR X $0.25/GAL.				$ 39,000
Harbor Maint.				
(50,000 CY/100 CY/HR) X 1000 HP X 1/8 GAL/HP-HR X $0.25/GAL.				$ 16,000
Pipe Turning				$ 20,000
Personnel				$ 90,000
Maintenance				$ 30,000
Interest (6-5/8%)				$ 73,000
Amortization (10 yrs.)				$ 82,000
TOTAL ANNUAL COST				$ 350,000

operating during periods when it is needed the most. This would necessitate creation of a storage area by cutting the west beach back by overdredging, which is environmentally and socially undesirable. The craters on the beach would pose a safety hazard to beach users, and they might undermine the jetty. The crane on the west jetty would be aesthetically displeasing and noisy. The reliability is judged medium because the eductors are portable, facilitating cleaning debris from the jets. The system is also considered to have a medium degree of certainty because it is flexible enough to remove sand either from shoals in the channel or near the east jetty. Because of the inability of the system to work during severe storms, it is anticipated that perhaps as much as one month of channel shoaling could occur, resulting in closure of the harbor. This system has a very distinct advantage in that its components are salvageable and could be re-used in a more permanent bypass system.

E.44 Alternative 9 - Fixed Hydraulic - Zipper. The Zipper system proposed as a maintenance alternative can also be used as a bypass system. Figure E10 shows one of several possible pipeline arrangements and table E12 is a cost estimate. This alternative has a low certainty because the success of the system relies heavily on the hypothesis that the primary source of shoaling is sand that bypasses the head of the west jetty. The certainty could be upgraded to medium by installing manifold intakes in the channel to remove shoals that accrete from other sources, but that is basically the maintenance alternative. This bypass system is less efficient and less reliable than the maintenance alternative, but it also has fewer adverse environmental effects than some of the other systems. The beach would not have to be cut back as much as in alternative 8 to provide storage because this system can operate during storm conditions whereas the mobile hydraulic system cannot. The harbor could be open for an estimated 11-1/2 months of the year, being closed only after storms when the infilling would greatly exceed the pumping capacity.

E.45 Alternative 10 - San Lorenzo River Sediment Trap. The San Lorenzo River, 3,000 feet updrift from the harbor, is believed to be a source of sediments, although detailed sedimentation rating curves for it have not been calculated. The river discharges an estimated 18,000 to 27,000 cubic yards of beach

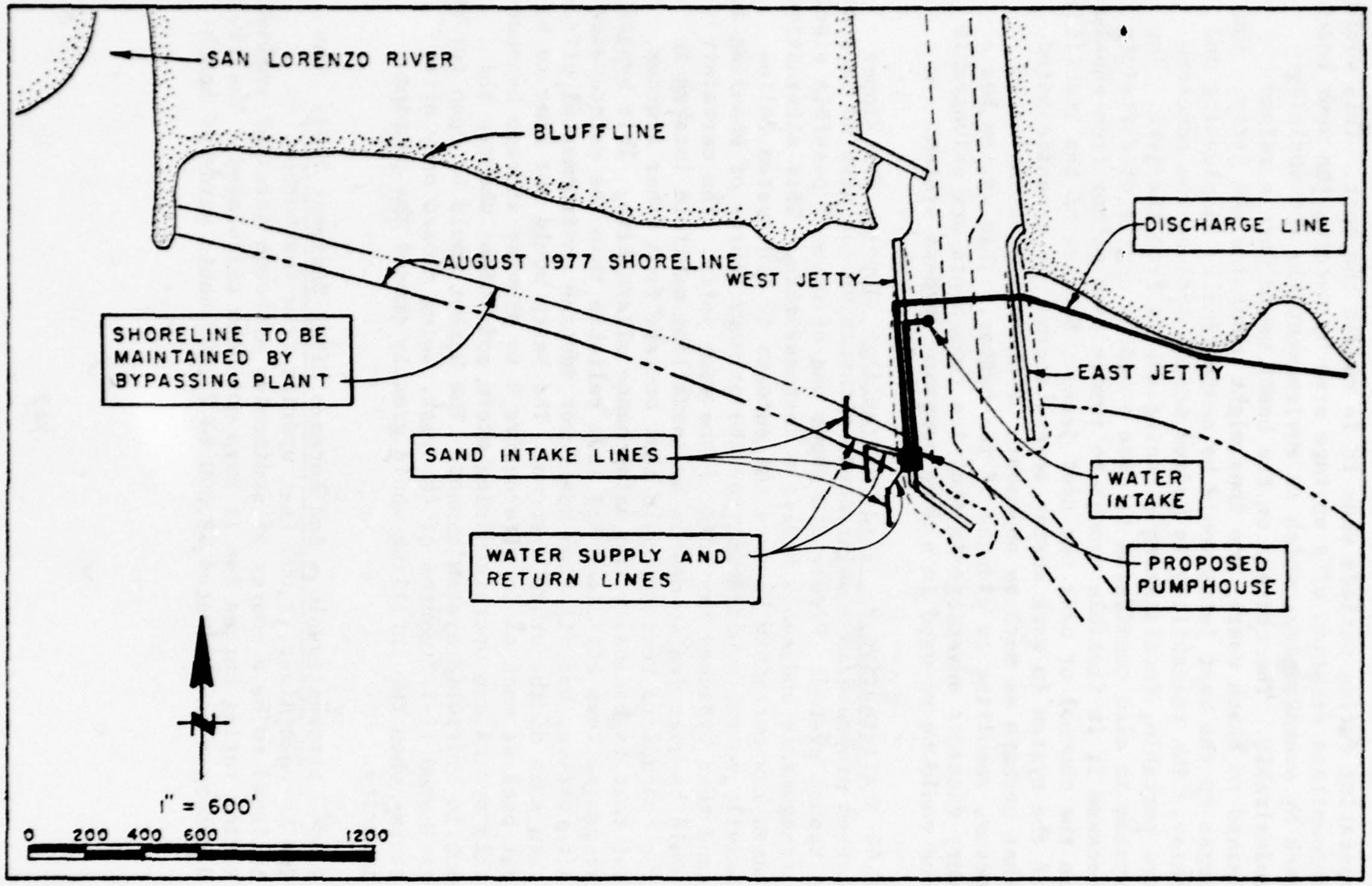

Figure E10. Zipper Bypassing

TABLE E12

COST ESTIMATE - Alternative #9

Zipper Bypassing System

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
FIRST COST				
Pumphouse (Inside of West Jetty-See Alt.6)				
Stone Removal		1	LS	$ 10,000
Bed Grout				$ 50,000
Concrete				$ 52,000
Roof				$ 20,000
Subtotal				$ 132,000
Sand Intake Lines (3)				
Manifold Line 20"Ø - Buy	450	LF	200	$ 90,000
Place Intake Lines	660	LF	350	$ 231,000
Jet System-Buy & Place	450	LF	125	$ 56,250
Slide Valve	3	EA	5,000	$ 15,000
Pipe Runs to Pump	3	EA	7,000	$ 21,000
Subtotal				$ 413,000

TABLE E12 (Continued)

Item	Quantity	Unit	Unit Price	Cost Estimate
Water Intake System				
12"Ø Pipe Run to BW (Buy, Place & Bury)	50	LF	90	$ 4,500
Riser Section	1	EA	LS	$ 4,000
Top of BW Pipe Run	500	LF	50	$ 25,000
Anchors @ 10'	50	EA	350	$ 17,500
Intake Struc.				$ 15,000
Subtotal				$ 66,000
Pumping Equipment				
750 HP Pump	1	EA	LS	$ 20,000
750 HP Motor	1	EA	LS	$ 30,000
Controls			LS	$ 100,000
Misc. Piping & Installation	62			$ 60,000
Connect Power				$ 45,000
Subtotal				$ 255,000
Discharge Line (18"Ø)($150,000 for 10"Ø)			LS	$ 200,000
Pipe Run Across Channel	350	LF	90	$ 31,500
Risers	2	EA	LS	$ 8,000
Subtotal				$ 240,000

TABLE E12 (Continued)

Item	Quantity	Unit	Unit Price	Cost Estimate
Construction Contingencies (25%)				$ 276,000
Subtotal				$1,382,000
Engineering & Design (10%)				$ 138,000
Supervision & Admin. (7%)				$ 97,000
TOTAL FIRST COST				$1,617,000
ANNUAL COST (See Alt. 6)				
Power (850 Hr X 750 HP X $0.055/KW-HR				$ 36,000
Maintenance				$ 50,000
Pipe Turning				$ 20,000
Personnel				$ 50,000
Interest (6-5/8%)				$ 107,100
Amortization (10 yrs.)				$ 119,100
Royalty	300,000	CY	$0.20	$ 60,000
TOTAL ANNUAL COST				$ 442,000

size material per year into littoral zone. Wave energy transports the coarser material downdrift into the harbor area and moves the fines offshore. The river could be used as a trap for flood borne sediments and for littoral drift transported to the river mouth area from updrift beaches by wave action. The lower reaches of the river could be dredged to provide a protected area where a dredge could be stationed to remove these deposits as they accumulate and to pump them to the disposal area east of the harbor.

E.46 Figure E11 shows a scheme based on using the San Lorenzo River as a sediment trap. Initially, 200,000 yards would be removed from the river to create the trap. Annual bypassing from the trap area and channel maintenance dredging at the harbor would total approximately 400,000 cubic yards per year. The 100,000 cubic yards in excess of the net littoral-drift rate of 300,000 yards would be due to fines in the river bed that are normally carried into the offshore region by the winnowing action of waves. The cost estimate is given in table E13.

E.47 This alternative relies heavily on the theory that most of the littoral drift will enter the river channel. The concept has a very low reliability because the river would be an inefficient trap, and shoaling could occur in the harbor from other transport mechanisms. The large delta offshore of the river mouth provides a mechanism to bypass littoral drift past the river mouth. Rivers have highly variable sedimentation rates from year to year. The river trap might work well during years of low discharge but would not be effective during years of excessive river flooding.

E.48 The beach between the river and the harbor would take several years to adjust to a point where it ceases to be a source of harbor shoaling. This would extend the length of time that the channel would be closed or necessitate pre-dredging the west beach. The certainty of the system is low because the success of the concept depends heavily on the hypothesis that the harbor-shoaling mechanism is primarily littoral transport from the updrift beaches. Depositing fine river sediments on the downdrift beach would degrade the quality of sand. Clearing of the river basin, however, would be aesthetically pleasing and beneficial to the flood control project. Removal of the material from the channel has several benefits that are not necessarily related to the shoaling problem.

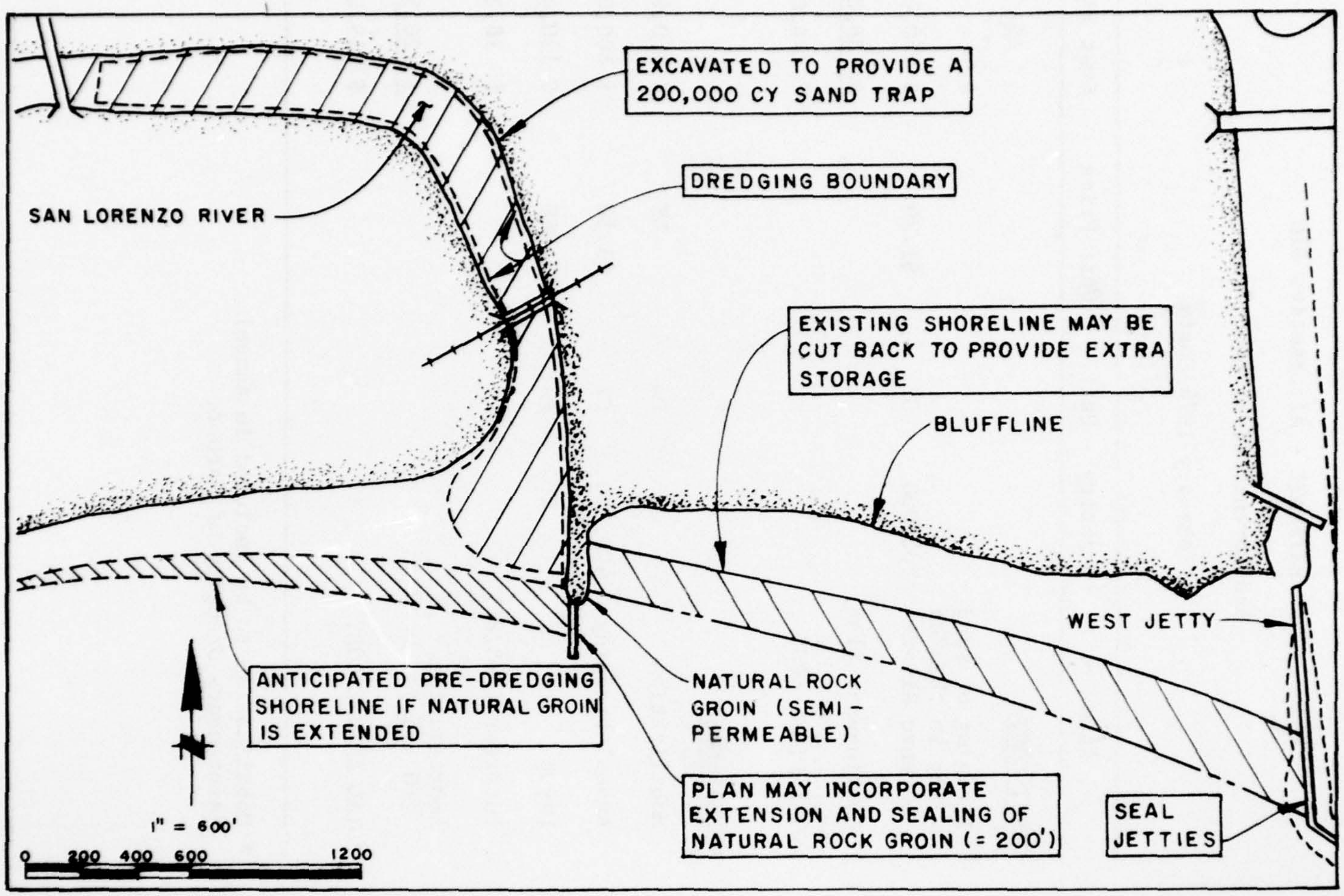

Figure E11. San Lorenzo River Sediment Trap

TABLE E13

COST ESTIMATE - Alternative #10

San Lorenzo River Sand Trap

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
FIRST COST				
Dredging of Sand Trap in the San Lorenzo River*	200,000	CY	$1.20	$ 240,000
Contingency (15%)				$ 36,000
TOTAL FIRST COST				$ 276,000
ANNUAL COST				
Mobilization	1	FA	LS	$ 150,000
Annual Dredging	400,000	CY	$1.50	$ 600,000
Layup	2	EA	$65,000	$ 130,000
Interest (6-5/8%)				$ 18,300
Amortization (10 yrs.)				$ 20,300
TOTAL ANNUAL COST				$ 919,000

* Mobilization to be included in Annual Maintenance Dredging of Harbor.

STRUCTURAL SYSTEMS

E.49 General. The maintenance dredging and bypassing alternatives were based on the assumption that the harbor configuration would not be changed. None of the solutions are anticipated to maintain the harbor open for navigation year round. The present harbor configuration provides neither adequate wave protection for a dredge nor adequate storage for high shoaling rates during severe storms. Modification of the harbor would be required to accomplish these purposes.

E.50 Sealing the Jetties. In the analysis of littoral processes it was estimated that approximately 20,000 cubic yards of littoral drift shoal in the channel in an average year by leaking through the voids in the armor and underlayer stone of the west jetty. Leakage through a quarry stone structure can be retarded by sealing the voids by pumping a thick, fast-setting concrete through holes drilled through the armor stone to seal the voids. Table E14 gives a cost estimate for sealing the seaward 800 feet of the west jetty. This is not an alternative solution in itself, but it does contribute to the solution. The effectiveness of the sealing depends upon which overall solution to the shoaling problem is adopted. Jetty sealing would not be necessary with the Zipper or the eductor maintenance systems because they are designed to handle material that leaks through the voids. Sealing the jetties alone would only partially reduce the quantity of sand that enters the harbor. Most of the material blocked from entering the harbor by sealing would merely be forced to bypass the head of the jetty. Part of that material would then enter the harbor and part would naturally bypass the channel over bars. Sealing the west jetty would be desirable for each bypassing alternative and for those maintenance alternatives that include periodic channel dredging. No method is available to measure accurately the quantity of sand passing through the voids. Prior to sealing the west jetty, a monitoring program should be implemented to determine the rate that littoral drift enters the harbor through the jetties. This could be accomplished by digging a trench on the shoulder of the navigation channel and sounding it to determine its rate of shoaling. The east jetty is shorter, has a higher core, is exposed to milder waves

TABLE E14

COST ESTIMATE - Sealing of West Jetty

Station 13+00 To 21+00

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
From 13+00 to 16+00				
Grouting - Buy & Place*	1200	CY	100	$ 120,000
From 16+00 to 21+00				
Grouting - Buy & Place*	500	CY	100	$ 50,000
Contingencies				$ 25,000
				$ 195,000

* Includes cost for drilling.

and is estimated to be a better littoral barrier than the west jetty. Therefore, sealing the east jetty is not considered necessary.

E.51 Alternative II - Offshore Breakwater - Annual Dredging. A proven solution for harbors along shorelines with high rates of littoral transport is to construct a breakwater offshore of the updrift beach designed to trap littoral drift and to provide protection for a large hydraulic dredge. This dredge would then periodically remove the accumulated sand. Examples of this solution on the West Coast are Channel Islands Harbor and Ventura Harbor.

E.52 Figure E12 shows an offshore breakwater at Santa Cruz designed to form a sediment trap in the updrift beach. The breakwater should be about 1,800 feet long and have a cross section as shown in figure E13. The armor units should be 8-ton dolos on the trunk and 10-ton dolos on the head to withstand a 22-foot design breaking wave. The sand trap is estimated to have a capacity to store approximately 900,000 cubic yards; therefore, a large hydraulic dredge would only have to be mobilized once every three years to bypass the accumulated sand. Table E15 is a cost estimate for this structural system, assuming that dredging would be required every three years.

E.53 The concept is highly reliable because the breakwater will alter the wave climate at the harbor entrance thereby reducing shoaling from several sources. Through careful analysis with the aid of model studies, the breakwater can be aligned to diffract waves in such a way that wave energy does not transport littoral drift into the harbor. Diffraction patterns shown in figure E12 illustrate this concept. Waves would tend to transport sand along the west jetty and form a spit on the updrift beach away from the harbor entrance. The breakwater would be so positioned that waves in the entrance channel would have about one-third the height they would have without the structure. Leakage of sand through voids in the west jetty and onshore transport would also be reduced due to the change of wave climate. Although it would be desirable to move the breakwater farther east to provide more protection for the harbor, this would move the trap too close to the entrance channel. The certainty of the system is high because the small amount of material that

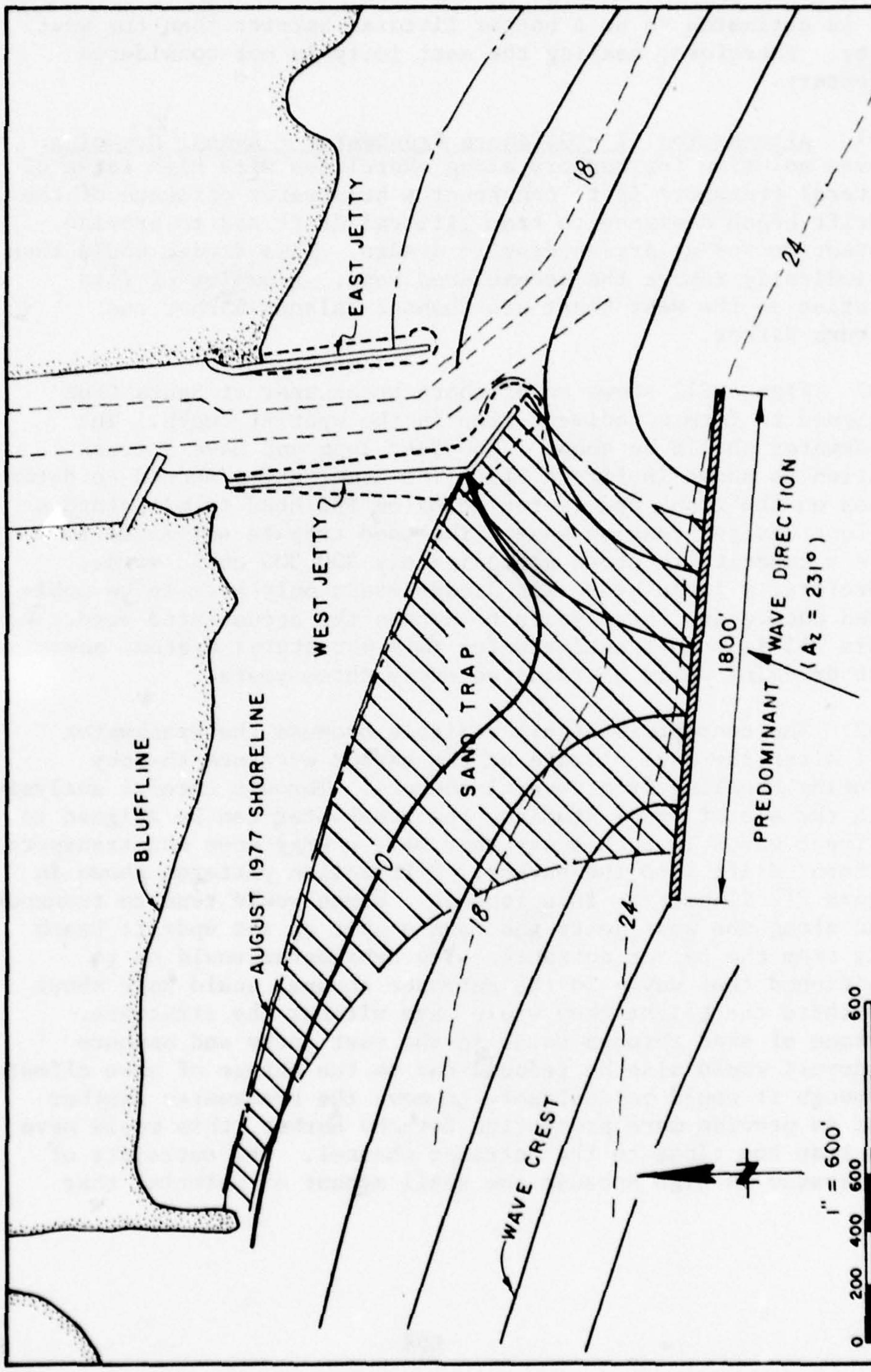

Figure E12. Offshore Breakwater

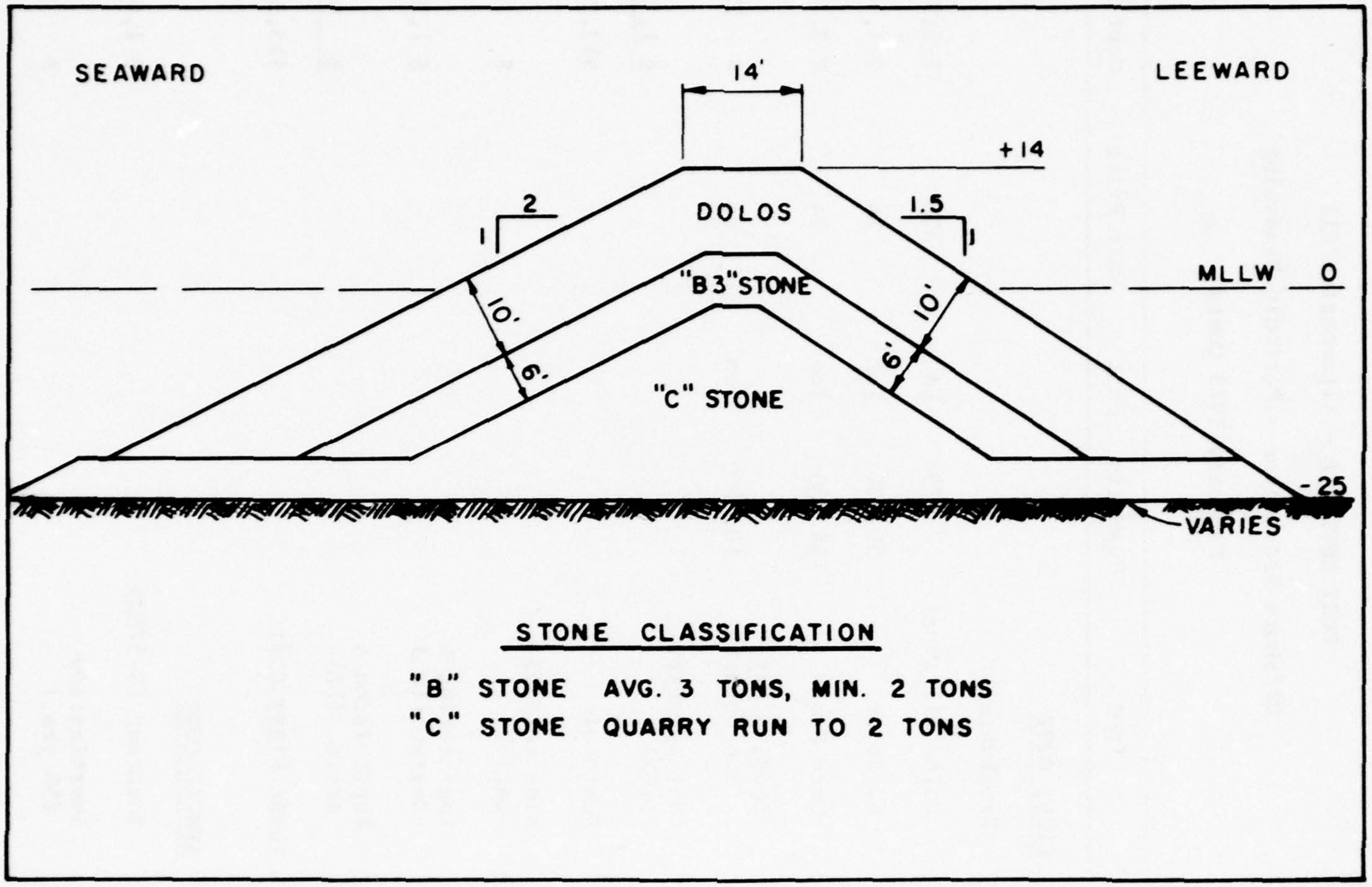

Figure E13. Offshore Breakwater Section

TABLE E15

COST ESTIMATE - Alternative #11

Offshore Breakwater - Periodic Bypassing

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
FIRST COST				
Breakwater				
Dolos (11 tons)	7,225	EA	830	$ 5,997,000
B3 Stone	56,200	Ton	26	$ 1,461,000
Core Stone	158,000	Ton	24	$ 3,792,000
Bedding Layer (5.6 tons/LF)	10,100	Ton	25	$ 252,000
Contingencies (15%)				$ 1,725,000
Subtotal				$13,227,000
Aids to Navigation				$ 35,000
Engineering & Design (10%)				$ 1,323,000
Supervision & Admin. (7%)				$ 926,000
TOTAL FIRST COST				$15,511,000
ANNUAL COST				
Interest (6-5/8%)				$ 1,028,000
Amortization (50 yrs.)				$ 43,000

TABLE E15 (Continued)

Item	Quantity	Unit	Unit Price	Cost Estimate
Dredging (Every 3 yrs.)				
Mobilization	1/3	LS	225,000	$ 75,000
Dredging (900,000/3)	300,000	CY	$ 1.20	$ 360,000
TOTAL ANNUAL COST				$ 1,506,000

escapes the trap and that which enters the channel from other sources can be removed during the periodic dredging episode.

E.54 The shoreline along the updrift beach will widen as far as the mouth of the San Lorenzo River because of the groin-effect of the trapped sand; however, the beach-berm height should not increase. The slightly wider beach would have a negligible influence on the effectiveness of the flood control project.

E.55 Bypassing large quantities of material infrequently may produce adverse environmental effects. The downdrift beaches would temporarily lose their source of sand, then large quantities of sand would be bypassed in a short period of time. If the material were bypassed after the winter waves that naturally transport littoral drift around Black Point, the east beach would build considerably. Under these conditions, updrift movement would tend to shoal the channel at a much greater rate than under present conditions. Each of these effects can be remedied by annual in lieu of triennial dredging at a slight increase in annual cost due to more frequent mobilization and demobilization charges.

E.56 Alternative 12 - Short Offshore Breakwater - Continuous Bypassing. The principal advantages of an offshore breakwater are that it provides protection for a dredge and increases the storage capacity for littoral drift. The primary disadvantage of an offshore breakwater, aside from its high initial cost, is the adverse environmental effect of beach and cliff erosion that could result from intermittent bypassing. Downdrift beaches would alternately be undernourished for two years and overnourished for one year. Considering that there appears to be a much greater energy potential for littoral transport than there is littoral material to be moved, this could result in erosion of the cliffs to the east. These effects could be mitigated by combining the offshore breakwater concept with a continuous sand-bypassing system. If continuous bypassing were implemented, a smaller storage capacity and hence a shorter breakwater would be required.

E.57 Figure E14 shows a 1000-foot offshore breakwater with a 100,000 cubic yard trap. The breakwater is in shallower water (-20 feet MLLW) than in Alternative 11. Several methods of bypassing are possible. The cost estimate in table E16 is based on bypassing with a semi-permanent eductor system. The eductor could be handled with shore-based equipment and could be serviced more readily than the eductor systems proposed for the maintenance and bypassing systems in Alternatives 5 and 7.

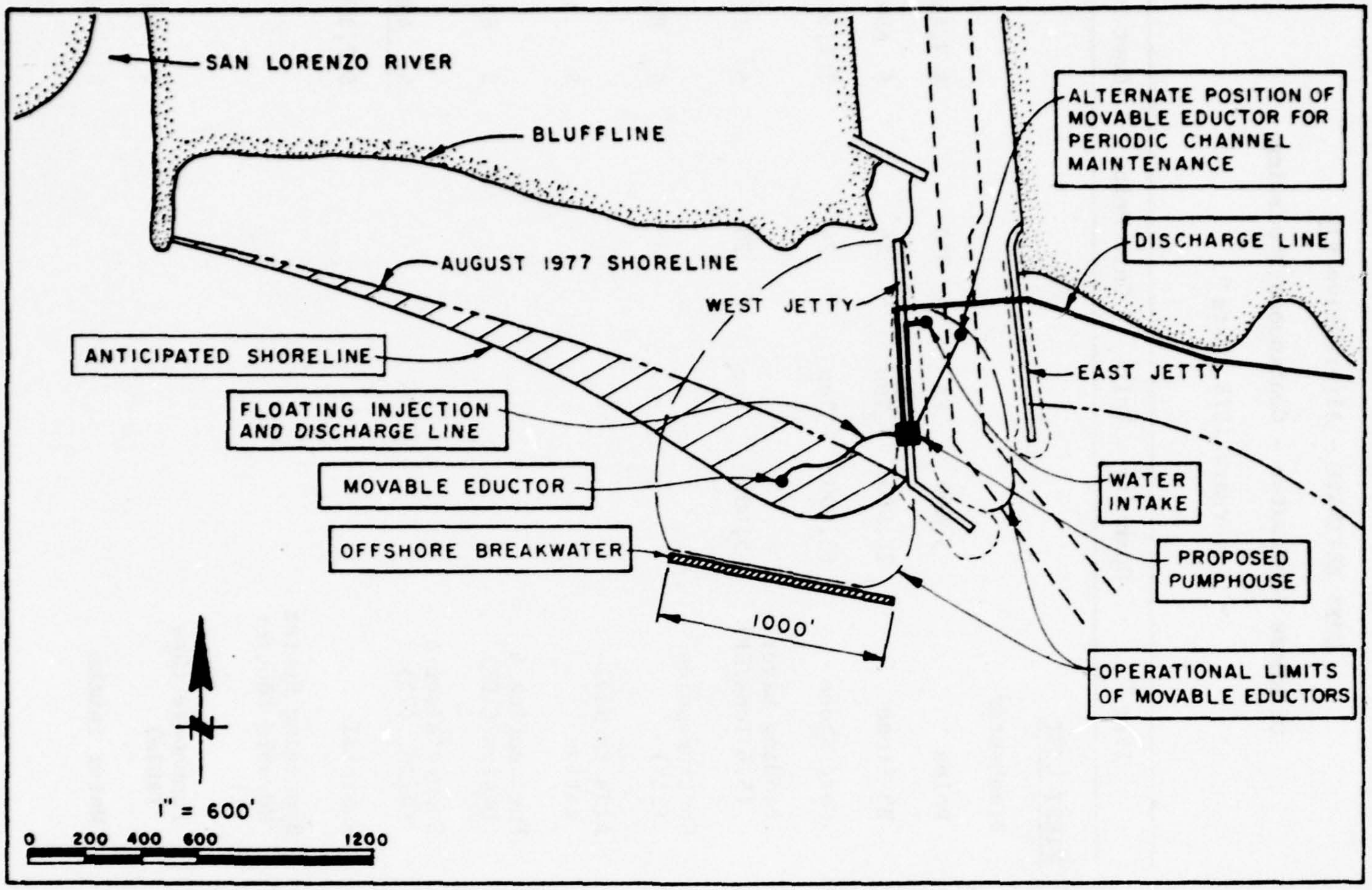

Figure E14. Offshore Breakwater Continuous Bypassing

TABLE E16

COST ESTIMATE - Alternative #12

Offshore Breakwater - Continuous Bypassing

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
FIRST COST				
Breakwater				
Dolos	3,800	EA	830	$ 3,154,000
B3 Stone	25,000	Ton	26	$ 650,000
Core Stone	63,000	Ton	24	$ 1,512,000
Bedding Layer (5.6 Tons/LF)	5,000	Ton	25	$ 125,000
Contingencies (15%)				$ 816,000
Aids to Navigation				$ 35,000
Engineering & Design (10%)				$ 626,000
Supervisison & Admin. (7%)				$ 438,000
Subtotal				$ 7,356,000
Bypassing System Movable Eductor (1)				
Pumphouse (See Table)				$ 132,000
Water Intake				$ 66,000

TABLE E16 (Continued)

Item	Quantity	Unit	Unit Price	Cost Estimate
Pumping Equip. & Jets				$ 40,000
Connect Power				$ 45,000
Injection & Return Lines	2,000	LF	50	$ 100,000
Floats	25	EA	250	$ 6,250
Riser	1	EA	LS	$ 4,000
Subtotal				$ 393,000
Discharge Line (See Table 9)				$ 177,000
Contingencies (25%)				$ 142,000
Engineering & Design (10%)				$ 71,000
Supervision & Admin. (7%)				$ 50,000
Subtotal				$ 440,000
TOTAL FIRST COST				$ 8,189,000

TABLE E16 (Continued)

Item	Quantity	Unit	Unit Price	Cost Estimate
ANNUAL COST				
Power (300,000/150 CY/HR) X 750 HP X $0.055/KW-HR				$ 82,000
Maintenance				$ 50,000
Pipe Turning & Replacement				$ 40,000
Personnel				$ 50,000
Interest (6-5/8%)				$ 545,000
Amortization				
Breakwater (50 Yrs.)				$ 20,000
Bypassing System (10 yrs.)				$ 64,000
TOTAL ANNUAL COST				$ 851,000

A portable eductor could also be extended into the harbor to remove shoals that form as a result of mechanisms other than bypassing of the west jetty.

E.58 The short offshore breakwater would not provide additional protection to the harbor, as its east end would have to be farther west than that of the longer breakwater in Alternative 11 in order to keep the trap well updrift of the navigation channel and to create the desired wave diffraction pattern. The concept is rated high in reliability and certainty and should maintain a year-round channel.

E.59 Alternative 13 - Extend West Jetty. The storage capacity of the west jetty could be increased by extending it seaward. The primary advantage of this concept is that additional storage can be provided without cutting back existing beaches. In order to be effective, this structural solution must be supplemented with some type of sand-bypassing system. A 400-foot jetty extension shown in figure E15 would provide storage capacity for approximately 500,000 cubic yards between the west jetty and the San Lorenzo River and approximately 15,000 cubic yards on the east side of the west jetty. Several bypassing systems could be used; however, the mobile hydraulic system of Alternative 8 was selected a being the most reliable and effective. The existing and proposed jetties should be sealed for this system to function properly. The cost estimate is given in table E17.

E.60 Extending the jetty reduces bypassing requirements during the first year or two. When the beach aggrades sufficiently however, sand would naturally bypass the end of the jetty at the same rate as under present conditions unless intercepted by the artificial bypassing system. The primary advantage of this alternative is that updrift beaches would be preserved and the shoal would form farther offshore, outside the navigation channel. The system has a high reliability but medium certainty. Certainty is reduced because the system addresses only the primary source of channel shoaling and does not prevent shoaling from littoral-drift reversals or from onshore transport of off-entrance deposits.

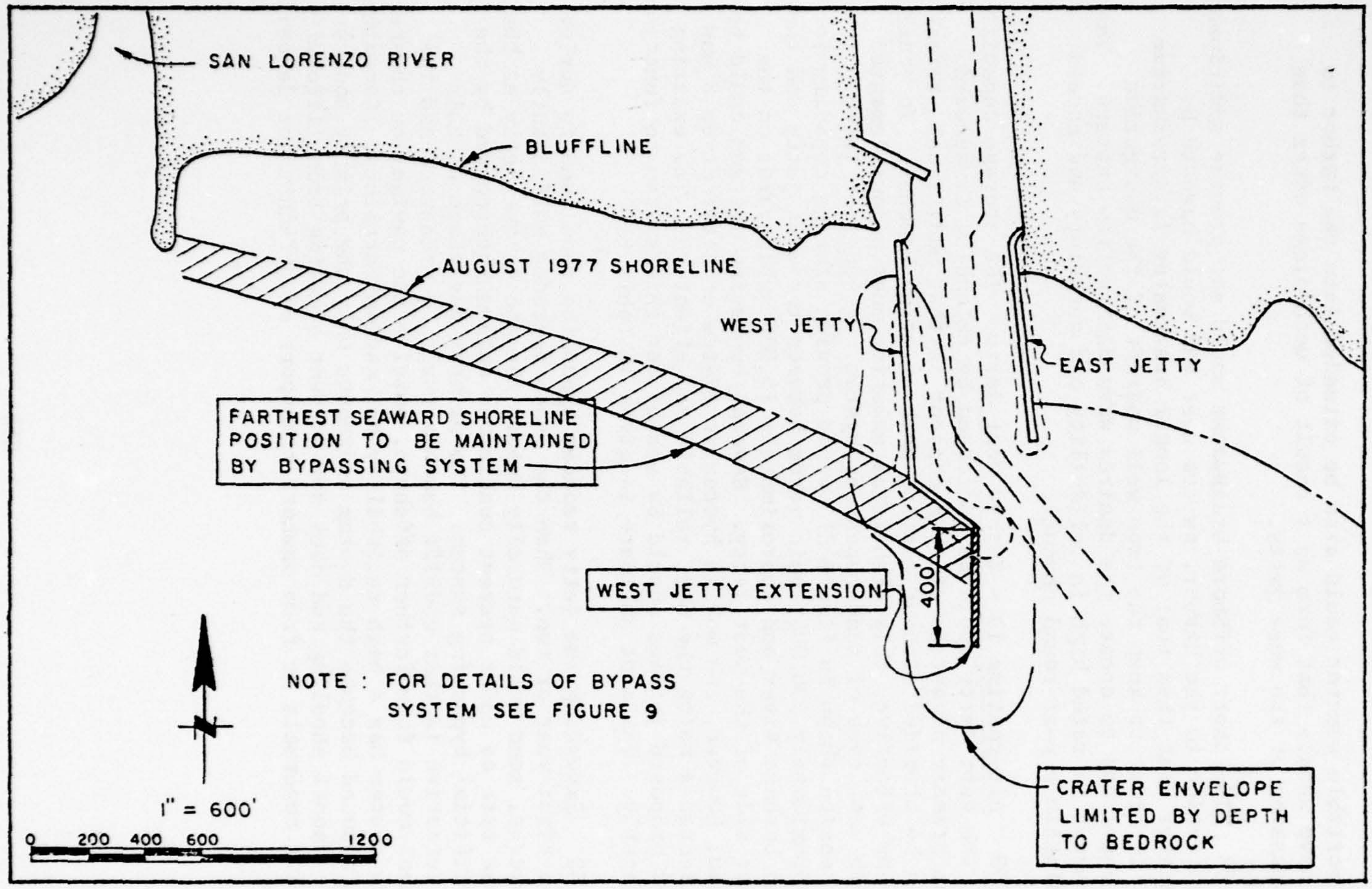

Figure E15. Extend West Jetty

TABLE E17

COST ESTIMATE - Alternative #13

Extension of West Jetty-Continuous Bypassing

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
FIRST COST				
Jetty Extension (400')				
Dolos	1,300	EA		$ 1,079,000
A10 Stone	8,731	Ton		$ 244,000
B3 Stone	24,622	Ton		$ 640,000
Core Stone	51,650	Ton		$ 1,240,000
Bedding Layer	2,888	Ton		$ 72,000
Concrete Cap	533	CY		$ 39,000
Concrete Grout	767	CY		$ 74,000
Seal Exist. West Jetty (See Table 14)				$ 170,000
Contingencies (15%)				$ 534,000
Aids to Navigation				$ 25,000
Engineering & Design (10%)				$ 409,000
Supervision & Admin. (7%)				$ 286,000
Subtotal				$ 4,787,000

TABLE E17 (Continued)

Item	Quantity	Unit	Unit Price	Cost Estimate
Mobile Eductor System (See Table 11)				
Additional Equipment				$ 440,000
Pickup Flanges	6	EA	250	$ 1,500
Supply & Discharge Line	800	LF	50	$ 40,000
Contingencies (25%)				$ 120,000
Engineering & Design (10%)				$ 60,000
Supervision & Admin. (7%)				$ 42,000
Subtotal				$ 704,000
TOTAL FIRST COST				$ 5,491,000
ANNUAL COST				
Fuel (300,000 CY/200 CY/HR) X 1000 HP x 1/8 GAL/ HP-HR X $0.25/GAL.)				$ 47,000
Pipe Turning				$ 20,000
Personnel				$ 90,000
Maintenance				$ 30,000
Interest (6-5/8%)				$ 364,000
Amortization				
Jetty Extension (50 yrs.)				$ 13,000
Bypassing Sys. (10 yrs.)				$ 52,000
TOTAL ANNUAL COST				$ 616,000

E.61 Alternative 14 - Modify Both Jetties or Construct a New Entrance. The narrow Santa Cruz entrance channel has very low storage capacity for littoral drift. Littoral drift that bypasses the head of the west jetty presently forms a tip shoal in the navigation channel. The storage capacity could be enlarged by widening the channel. Removal of the west jetty seaward of the angle point and extending the jetty seaward would be very expensive and was not considered because the same effects could be obtained by just adding structures.

E.62 Several entrance configurations that would improve the harbor from both maintenance and navigational viewpoints were considered. Figure E16 shows a modification of the entrance that would create a storage area and provide room for the tip shoal to form outside the project channel. This unusual configuration is a modified arrowhead breakwater system. The proposed new east jetty would reduce channel shoaling by the mechanism of reversals in littoral drift and onshore transport. The arrowhead configuration also provides secondary benefits of wave attenuation in the harbor. Table E18 gives the cost estimate of the proposed jetties. Littoral drift would have to be bypassed or removed by maintenance dredging. Several of the maintenance or bypassing systems mentioned in Alternatives 1 through 10 could be used effectively to maintain the channel. The storage capacity of the protected area would be increased as much as five times over the existing capacity, thereby increasing the length of time it takes for the channel to shoal. The most cost-effective system would be annual dredging by a large hydraulic dredge; however, the cost estimate is based on implementing a phased dredging program which would have higher reliability and certainty. The greater storage capacity would allow the dredge to mobilize later in the year and demobilize earlier, thereby reducing the time required for the dredge to be on the site. The added wave protection would allow the dredge to operate a greater percentage of the year. These factors would improve navigation and would reduce annual dredging costs.

E.63 Alternative 15 - Weir Jetty or Groin - Continuous Bypassing. A weir jetty is a semi-permeable jetty which traps sediment in a sheltered area and maintains the updrift beach profile. Weir

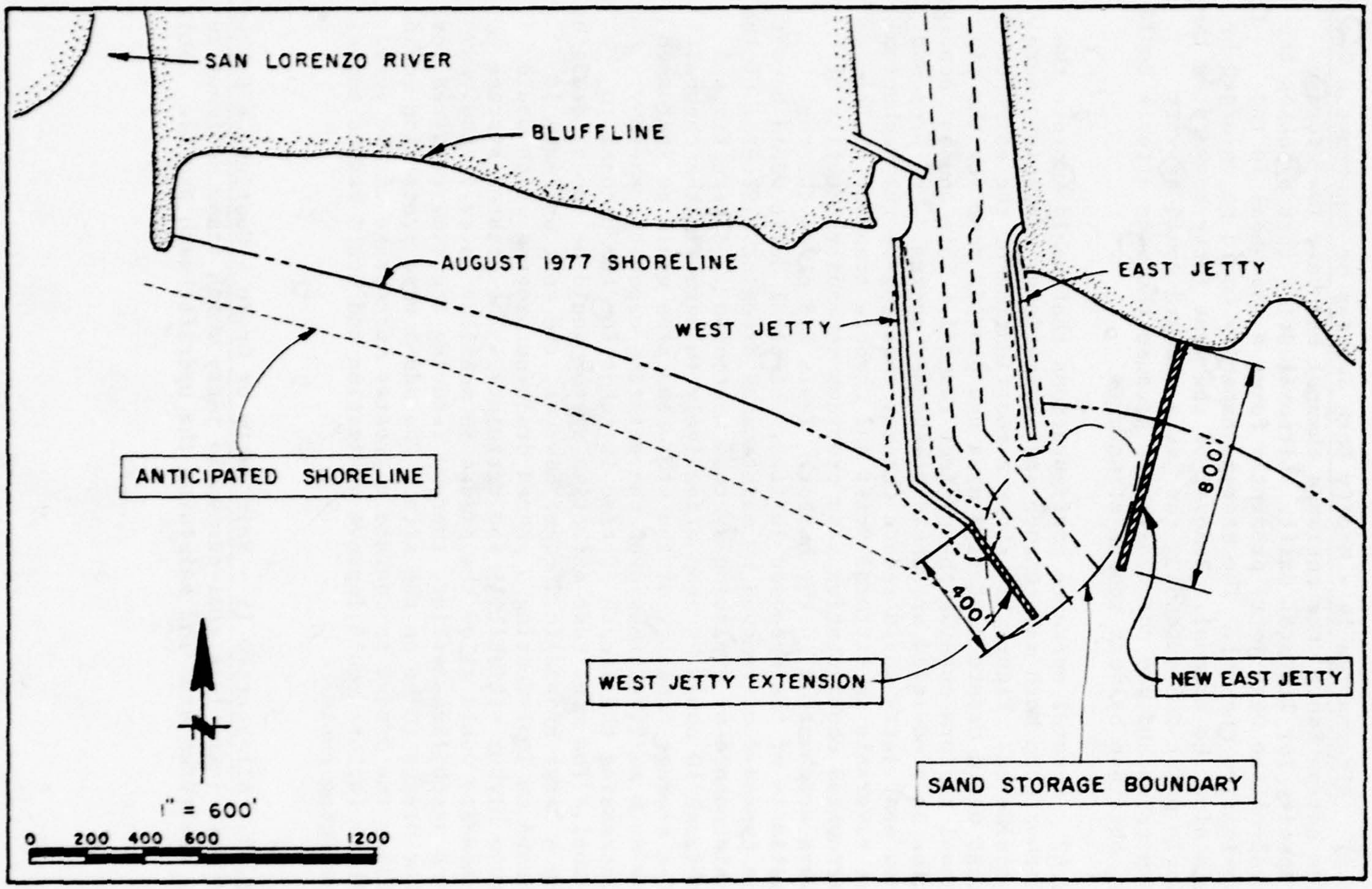

Figure E16. Modify Entrance

TABLE E18

COST ESTIMATE - Alternative #14

Modify Both Jetties

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
FIRST COST				
West Jetty Extension (See Table 17)				$ 3,388,000
Seal Exist. West Jetty				$ 170,000
New East Jetty				
A10 Stone	8,500	Ton	28	$ 238,000
B3 Stone	20,200	Ton	26	$ 525,000
Core Stone	20,700	Ton	24	$ 497,000
Bedding Layer	4,500	Ton	25	$ 112,000
Grout	1,800	CY	96	$ 173,000
Subtotal				$ 5,103,000
Contingencies (15%)				$ 765,000
Aids to Navigation				$ 35,000
Engineering & Design (10%)				$ 587,000
Supervision & Admin. (7%)				$ 411,000
TOTAL FIRST COST				$ 6,901,000

TABLE E18 (Continued)

Item	Quantity	Unit	Unit Price	Cost Estimate
ANNUAL COST				
Mobilization (Jan-Apr 15)	1		LS	$ 150,000
Layup	1	Month	65,000	$ 65,000
Dredging	275,000	CY	1.50	$ 412,000
Interest (6-5/8%)				$ 457,800
Amortization (50 yrs.)				$ 19,000
TOTAL ANNUAL COST				$ 1,103,000

jetties constructed at Masonboro, Ponce de Leon, and East Pass Inlets have not been totally effective. Experiences are limited and concepts are being further developed, but consideration is given here to weir-jetty and weir-groin systems.

E.64 One possible method of constructing a weir jetty is to expand the harbor boundaries as shown in figure E17. An offshore breakwater connected to the west jetty would be required to trap sand and to provide protection for a floating plant. A low-sill or weir section with a crown elevation along the desired updrift beach profile, would permit wave action to fill the trap area without eroding the updrift beach. A floating plant would be mobilized to perform maintenance and bypass dredging. Table E19 is a cost estimate based on pumping 300,000 cubic yards per year.

E.65 This system is rated low in reliability because it is not proven and low in certainty because the system is primarily effective only in bypassing littoral drift from the west beach to the east beach. It does not prevent shoaling of the channel by transport-reversals and onshore-transport mechanisms. Advantages of this system would be that the dredge would have a protected area in which to work and that it could also be used for maintenance dredging. A smaller dredge would bypass sand more continuously than other systems, inducing a lesser impact on downdrift beaches. The sand-trap basin could also be used in summer as an anchorage for transient boats.

E.66 The weir jetty concept is not well suited to this site; however, by creating a trap with a weir groin instead of a weir jetty, costs can be lowered. Figure E18 shows a plan view of such a system. The weir groin constructed on the updrift beach would maintain the desired beach profile while an eductor system could continuously bypass sand that enters the trap. Permanent eductors would be positioned in excavations into the hardpan to form the primary sand trap. A portable eductor could be used to remove material from other areas surrounding the trap and from the channel as required. A cost estimate is given in table E20.

E.67 The concept is rated low in both reliability and certainty because fixed eductors have not proved successful and because

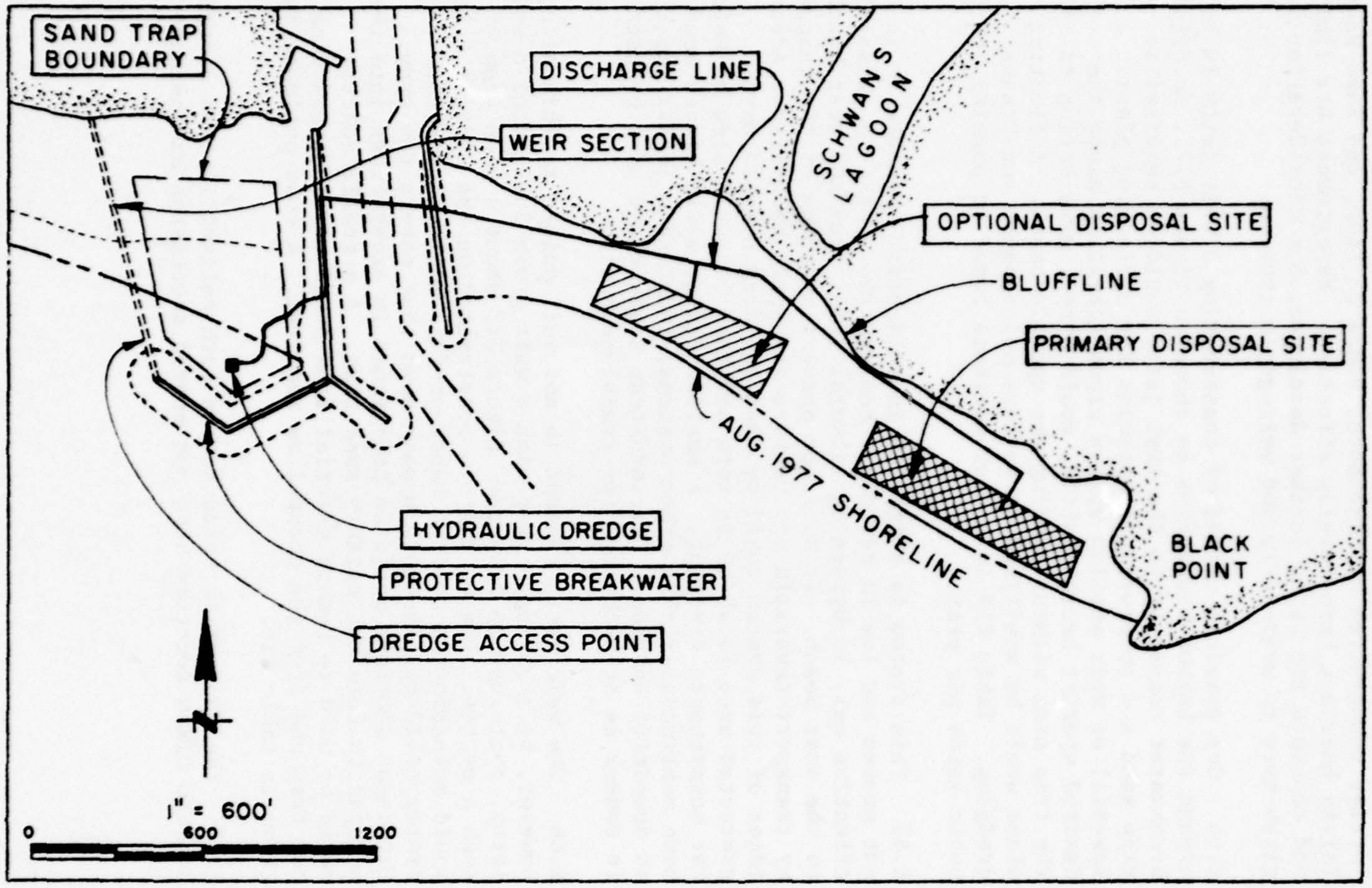

Figure E17. Weir Jetty

TABLE E19

COST ESTIMATE - Alternative #15a

Weir Jetty

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
FIRST COST				
Protective Jetty				
Dolos	1,450	EA	830	$ 1,204,000
A10 Stone	25,200	Ton	28	$ 706,000
B3 Stone	35,200	Ton	26	$ 915,000
Core Stone	29,500	Ton	24	$ 708,000
Bedding Layer (5.6 Ton/LF x 700 LF)	3,920	Ton	25	$ 98,000
Subtotal				$ 3,631,000
Weir Section				
Concrete Sheet Pile (Buy & Place)	900	LF	1,000	$ 900,000
Sand Trap-Dredging	150,000	CY	1.50	$ 225,000
Contingencies (15%)				$ 713,000
Aids to Navigation				$ 35,000
Engineering & Design (10%)				$ 547,000
Supervision & Admin. (7%)				$ 383,000
TOTAL FIRST COST				$ 6,434,000

TABLE E19 (Continued)

Item	Quantity	Unit	Unit Price	Cost Estimate
ANNUAL COST				
Mobilization	1		LS	$ 150,000
Dredging	300,000	CY	1.50	$ 450,000
Interest (6-5/8%)				$ 426,000
Amortization (50 yrs.)				$ 18,000
TOTAL ANNUAL COST				$1,044,000

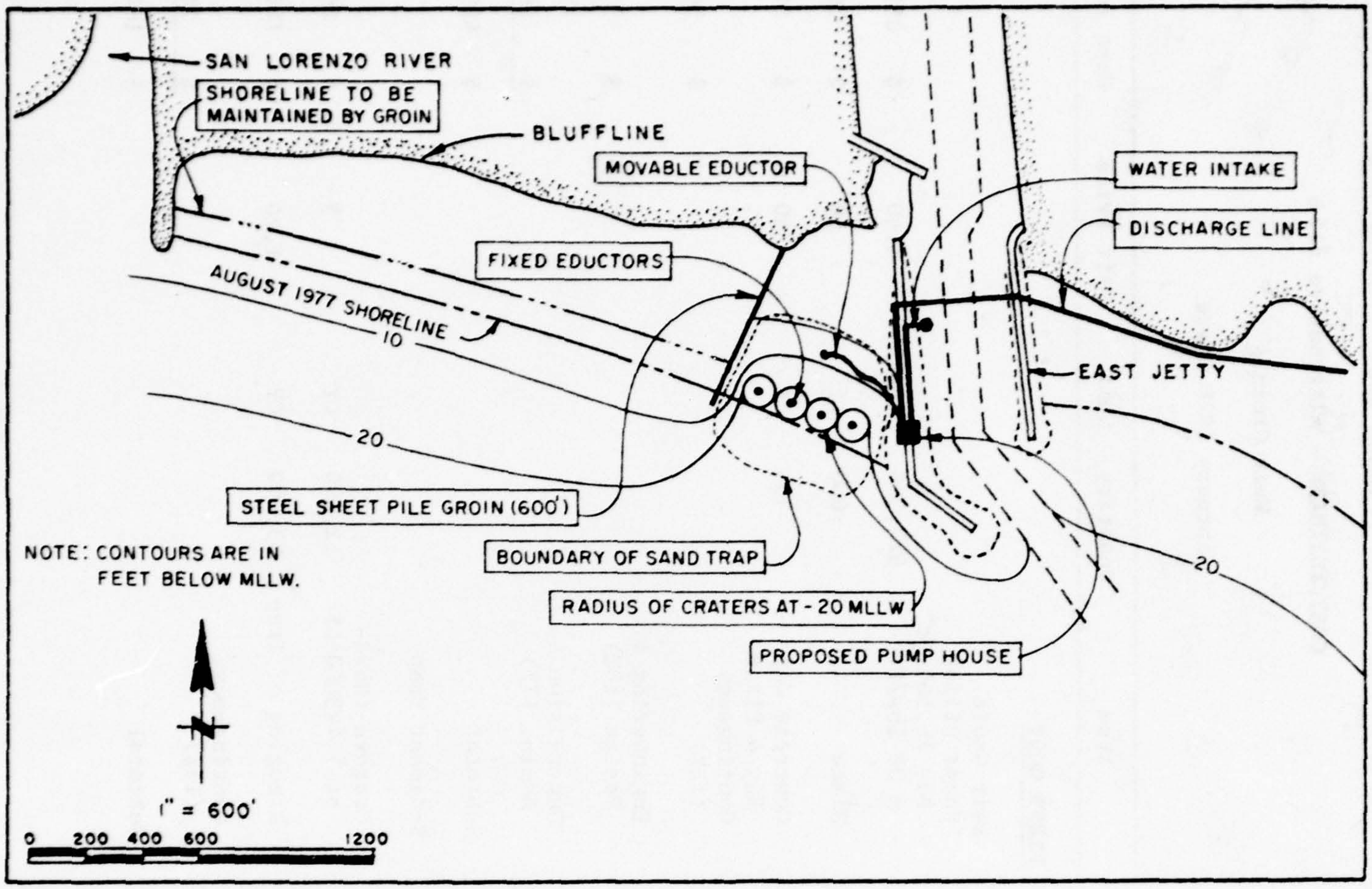

Figure E18. Weir Groin

TABLE E20

COST ESTIMATE - Alternative #15 b

Weir Groin

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
FIRST COST				
Weir Groin				
Sheet Piling				
Buy 16,500 ft^2 x 38 lbs/ft^2	627,000	lbs.	0.40	$ 251,000
Place	600	LF	50	$ 30,000
Concrete Cap Buy & Place	355	CY	250	$ 89,000
Contingency (15%)				$ 56,000
Engineering & Design (10%)				$ 37,000
Supervision & Admin. (7%)				$ 26,000
Subtotal				$ 489,000
Sediment Trap				
Craters (Hard-pan) 4x3000 CY	12,000	CY	5	$ 60,000
Dredging of Trap	83,000	CY	1.20	$ 100,000
Contingency (15%)				$ 24,000
Subtotal				$ 184,000

TABLE E20 (Continued)

Item	Quantity	Unit	Unit Price	Cost Estimate
Bypassing System-Eductor (4)				
Pumphouse (See Table 10)				$ 132,000
Water Intake Line				$ 66,000
Pumping Equip. & Jet Pumps				$ 195,000
Connect Power				$ 45,000
Discharge Line (See Table 10)				$ 177,000
Injection & Return Lines	3,200	LF	50	$ 160,000
Risers	4	EA	4,000	$ 16,000
Trenching & Connections				$ 100,000
Contingency (25%)				$ 223,000
Engineering & Design (10%)				$ 89,000
Supervision & Admin. (7%)				$ 62,000
Subtotal				$1,262,000
TOTAL FIRST COST				$1,938,000

TABLE E20 (Continued)

Item	Quantity	Unit	Unit Price	Cost Estimate
ANNUAL COST				
Power (300,000/150 CY/HR) x 750 HP x $0.055/KW-HR				$ 82,000
Maintenance				$ 50,000
Pipe Turning & Placement				$ 40,000
Personnel				$ 50,000
Interest (6-5/8%)				$ 128,000
Amortization				
Sheet Pile Groin (15 yrs.)				$ 20,000
Sediment Trap-Dredging (20 yrs.)				$ 5,000
Bypassing System (10 yrs.)				$ 93,000
TOTAL ANNUAL COST				$ 468,000

the system is heavily dependent on downdrift transport as being the primary source of shoaling. Proper operation of the eductor bypass concept, Alternative 7, should produce the same results.

E.68 Alternative 16 - Enhance Ebb Currents. Tidal currents naturally maintain coastal inlets as a function of the wave-energy flux and tidal prism. The tidal prism of the present harbor is 9.8 million cubic feet. Assuming a 5.3-foot mean diurnal tide range the harbor should maintain a 400-square-foot channel cross section due to natural tidal flushing. The small channel observed in Santa Cruz harbor after winter shoaling confirms this approximate figure. The cross-sectional area of the project channel in the throat is 3,900 square feet. A land area of 550 acres would be required to provide adequate tidal prism to maintain a channel of these dimensions using the O'Brien equation (1969). This land area is not available at the project site. Hydraulically connecting Schwanns Lagoon to the existing basin would add only 31 acres, which would have negligible influence on flushing.

E.69 Another concept would be to use tide gates to store water on flood tides and quickly discharge it during ebb tides creating high scouring velocities for a short duration. This would be hazardous to boaters. Ebb currents could be enhanced by pumping seawater into the basin to artificially duplicate the tidal prism. The tidal prism effect could be duplicated by creating an average flow of 3000 cfs for 12 hours a day. This would increase the peak velocity at project dimensions from about .2 feet per second to 1.2 feet per second. An 18-foot diameter pipe buried through the surf zone that discharged into the harbor, pumping at a pipe velocity of 11 feet per second, would be required. Thirty pumps, each with a discharge rate of 100 cfs, would be required to generate the flows. Table E21 is a cost estimate for this system.

E.70 This alternative would present several adverse impacts. The intake and discharge areas would be dangerous to boaters and swimmers. Enormous quantities of energy would be consumed. The noise of the pumps would be objectionable to residents. Although the system probably would work to some degree, it is not proven. The maximum currents would be about 1.2 feet per second which is below the steady-flow scouring velocity for

TABLE E21

COST ESTIMATE - Alternative #16

Enhance Ebb Currents

February 1978 Costs

Item	Quantity	Unit	Unit Price	Cost Estimate
PUMPS: (100 cfs)	30	CFS	10,000	$ 3,000,000
Contingencies (15%)				$ 450,000
Subtotal				$ 3,450,000
Engineering & Design (10%)				$ 345,000
Supervision & Admin. (7%)				$ 242,000
TOTAL FIRST COST				$ 4,037,000
Interest (6-5/8%)				$ 268,000
Amortization (10 yrs.)				$ 297,000
TOTAL ANNUAL COST				$ 565,000
DIESEL ENGINES:				$ 7,364,000
Contingencies (15%)				$ 1,104,000
Subtotal				$ 8,468,000
Engineering & Design (10%)				$ 847,000
Supervision & Admin. (7%)				$ 593,000
TOTAL FIRST COST				$ 9,908,000
Interest (6-5/8%)				$ 656,000
Amortization (30 yrs.)				$ 112,000
TOTAL ANNUAL COST				$ 768,000

TABLE E21 (Continued)

Item	Quantity	Unit	Unit Price	Cost Estimate
FIRST COST				
Pumps				$ 4,037,000
Diesels				$ 9,908,000
Intake Structure				$ 6,318,000
TOTAL FIRST COST				$20,263,000
ANNUAL COST				
Pumps				$ 565,000
Diesels				$ 768,000
Intake				$ 444,000
Fuel				$ 1,520,000
TOTAL ANNUAL COST				$ 3,297,000

sand. These low velocities may not be adequate to maintain the channel to project dimensions even with the stirring action of waves. Generally, channel velocities between 2 and 3 feet per second are required to scour sand in steady flow. The concept could be used to reduce infilling during storms, but even an order-of-magnitude less pumping and equipment would not be economically feasible because a dredge would still be required to clear the channel to project dimensions. Therefore, the reliability of the system is rated medium, the certainty is rated high because material would be removed from the channel independently of how it shoaled.

SUMMARY

E.71 General. Sixteen alternative solutions for mitigating the shoaling of Santa Cruz Harbor have been described and evaluated. Table E22 presents a summary of the costs, effectiveness, reliabilities, certainty, and impacts of each alternative. Figure E19 provides a method of determining cost effectiveness by plotting annual cost versus risk. Risk was defined as the inverse of the overall evaluation of reliability and certainty.

E.72 Review by Committee on Tidal Hydraulics. The shoaling processes and the alternative solution-studies were presented to the Committee on Tidal Hydraulics at Vicksburg, Mississippi, for discussion and evaluation. The committee evaluations were based on technical feasibility without consideration of cost. Table E23 summarizes the results of those evaluations. Alternative 2, implementation of the phased dredging program and Alternative 11, construction of a long offshore breakwater with harbor maintenance by a large floating plant, were most technically feasible.

ALTERNATIVE	MAINTENANCE OR BY-PASSING QUANTITIES (1000 CY.)	COST		EFFECTIVENESS			ENVIRONMENTAL IMPACTS	S
		FIRST	ANNUAL	MONTHS OPEN	RELIABILITY	CERTAINTY		
MAINTENANCE (CHANNEL)								
(1) ANNUAL DREDGING - FLOATING PLANT	100	NONE	300,000	8½	HIGH	HIGH	MINOR IMPACT ON BIOTA, PERIODIC SATURATION & STARVATION OF DOWNDRIFT BEACHES DUE TO PERIODIC DREDGING	SOC PAS THI
(2) PHASED DREDGING - FLOATING PLANT	200	NONE	580,000	11	HIGH	HIGH		GRE YEA WIN
(3) HOPPER DREDGE	200	2,018,000	444,000	10½	LOW	HIGH		NO RE
(4) MECHANICAL - DREDGING SYSTEMS	200	NONE	522,000	9	LOW	MEDIUM		TRU DAN
(5) FIXED HYDRAULIC SYSTEM - EDUCTOR	250	1,546,000	385,000	11½	LOW	MEDIUM	CONTINUOUS BYPASSING WILL CAUSE LEAST ADVERSE EFFECTS ON DOWNDRIFT BEACHES	MI
(6) FIXED HYDRAULIC SYSTEM - ZIPPER	250	1,517,000	407,000	11½	MEDIUM	MEDIUM		MI
BYPASSING (UPDRIFT)								
(7) FIXED HYDRAULIC SYSTEM - EDUCTOR	300	1,370,000	377,000	11½	LOW	LOW		
(8) MOBILE HYDRAULIC SYSTEM - EDUCTOR	300	1,107,000	350,000	11	MEDIUM	MEDIUM		CA S CO CO BE
(9) FIXED HYDRAULIC SYSTEM - ZIPPER	300	1,617,000	442,000	12	LOW	LOW		
(10) SAN LORENZO RIVER SEDIMENT TRAP	400	276,000	919,000	8	LOW	LOW	AIDS FLOOD CONTROL PROJECT. ENHANCES FISH RUNS.	IM OF
STRUCTURAL								
(11) OFFSHORE BREAKWATER - ANNUAL DREDGING	300	15,511,000	1,506,000	12	HIGH	HIGH	PERIODIC SATURATION & STARVATION OF DOWNDRIFT BEACHES DUE TO PERIODIC BYPASSING.	CR BA
(12) OFFSHORE BREAKWATER - CONTINUOUS BYPASSING	300	8,189,000	851,000	12	HIGH	HIGH	CONTINUOUS BYPASSING WILL CAUSE LEAST ADVERSE EFFECTS ON DOWNDRIFT BEACHES	CR TO
(13) EXTEND WEST JETTY	300	5,491,000	616,000	12	HIGH	MEDIUM	MINIMAL	PR DE
(14) MODIFY BOTH JETTIES - NEW ENTRANCE	275	6,901,000	1,103,000	12	HIGH	HIGH	MINIMAL	PR PR BA
(15) WEIR GROIN - CONTINUOUS BYPASSING	300	1,938,000	468,000	11½	LOW	LOW	CONTINUOUS BYPASSING WILL CAUSE LEAST ADVERSE EFFECTS ON DOWNDRIFT BEACHES.	EX BE
(16) ENHANCE EBB CURRENTS	—	20,263,000	3,297,000	12	MEDIUM	HIGH	HIGH ENERGY CONSUMPTION NOISY OPERATION.	DA AND

TABLE E22

EVALUATION OF ALTERNATIVES

...NTY	ENVIRONMENTAL IMPACTS	SOCIAL IMPACTS	NAVIGATION & HARBOR PROTECTION	COMMENTS
...N	MINOR IMPACT ON BIOTA, PERIODIC SATURATION & STARVATION OF DOWNDRIFT BEACHES DUE TO PERIODIC DREDGING	SOCIAL DISSATISFACTION DUE TO PAST IMPLEMENTATION OF THIS ALTERNATIVE	NAVIGATION MAY BE RESTRICTED DURING WINTER MONTHS BY THE FORMATION OF A TIP SHOAL	MOST EXPERIENCE
...N		GREATER POTENTIAL TO PROVIDE YEAR ROUND HARBOR DURING MILD WINTERS THAN ANNUAL DREDGING	NAVIGATION IS RESTRICTED DUE TO TIP SHOAL. PRESENCE OF DREDGE DOES NOT WORSEN SITUATION.	INITIATED PROCEDURE IN 1977-1978 SEASON
...N		NO EASEMENT OR PIPELINE REQUIRED.	MINOR HINDERANCE TO NAVIGATION.	REQUIRES CONSTRUCTION OF NEW HOPPER DREDGE
...UM	CONTINUOUS BYPASSING WILL CAUSE LEAST ADVERSE EFFECTS ON DOWNDRIFT BEACHES	TRUCK HAUL ON BEACH DANGEROUS & UNSIGHTLY	HINDERS NAVIGATION	EQUIPMENT TO OPERATE ON A RENTAL BASIS
...UM		MINIMAL	NO HINDERANCE TO NAVIGATION.	SYSTEM PRESENTLY UNDERGOING FIELD EVALUATION, EDUCTORS ARE SUSCEPTIBLE TO CLOGGING.
...UM		MINIMAL	NO HINDERANCE TO NAVIGATION	BYPASSING SYSTEM HAS 10 YEARS OPERATING EXPERIENCE AT ROSARITO BEACH, MEXICO
		CRATERS ON BEACH ARE SAFETY HAZARD. CONTINUOUS BYPASSING COULD REDUCE UPDRIFT BEACH WIDTH.	MINIMAL HINDERANCE TO NAVIGATION.	PRESENCE OF DEBRIS ON UPDRIFT WILL INCREASE POSSIBILITY OF SHUT DOWN OF BYPASSING PLANT DUE TO EDUCTORS CLOGGING.
...UM			MINIMAL HINDERANCE DURING MAINTENANCE PROCEDURE.	FLEXIBLE COMPONENT SYSTEM WITH RELATIVELY HIGH SALVAGE VALUE
			NO HINDERANCE TO NAVIGATION.	ADDITION OF 2 MAINTENANCE PIPES WOULD ADD AN ADDITIONAL $ 344,000 TO FIRST COST.
	AIDS FLOOD CONTROL PROJECT. ENHANCES FISH RUNS.	IMPROVE APPEARANCE OF CHANNEL	NO HINDERANCE TO NAVIGATION	A 200' EXTENSION OF NATURAL GROIN WOULD ADD AN ADDITIONAL $ 600,000 FIRST COST & WOULD INCREASE EFFICIENCY OF TRAP TO A SMALL DEGREE
	PERIODIC SATURATION & STARVATION OF DOWNDRIFT BEACHES DUE TO PERIODIC BYPASSING.	CREATES A SHELTER FOR BATHERS	PROTECTS HARBOR FROM LARGE NW SWELL, BUT OFFERS NO PROTECTION FROM LOW SE SEAS. DOES NOT HINDER NAVIGATION	PROVEN EFFECTIVE AT CHANNEL ISLAND HARBOR
	CONTINUOUS BYPASSING WILL CAUSE LEAST ADVERSE EFFECTS ON DOWNDRIFT BEACHES	CRATERS DANGEROUS TO BATHERS	MINOR HINDERANCE TO NAVIGATION	ALTERNATIVE ALLOWS A MORE PERMANENT BYPASSING SYSTEM TO BE INCLUDED IN THE FUTURE
...M	MINIMAL	PRESERVES UPDRIFT BEACHES.	PROTECTS HARBOR FROM LARGE NW SWELL. SHOAL STARTS TO FORM OUTSIDE OF PROJECT CHANNEL	WAVE PROTECTION COULD BE IMPROVE WITH A DIFFERENT ALIGNMENT OF THE JETTY EXTENSION.
	MINIMAL	PRESERVES UPDRIFT BEACHES. PROVIDES SHELTER FOR BATHERS.	MINOR DISTURBANCE TO NAVIGATION DURING DREDGING. PROTECTS HARBOR FROM LARGE NW SWELL. SHOAL STARTS TO FORM OUTSIDE OF PROJECT CHANNEL	INCREASED SAND STORAGE AT ENTRANCE MAY ALLOW ANNUAL DREDGING TO REPLACE PHASED DREDGING REDUCING COST.
	CONTINUOUS BYPASSING WILL CAUSE LEAST ADVERSE EFFECTS ON DOWNDRIFT BEACHES.	EXPROPRIATES PUBLIC BEACHES.	NO HINDERANCE TO NAVIGATION	THE SAME RESULTS CAN BE OBTAINED WITH FIXED BYPASSING AT LESS COST.
	HIGH ENERGY CONSUMPTION NOISY OPERATION	DANGEROUS AREA NEAR INTAKE AND DISCHARGE STRUCTURES.	MINIMAL EFFECTS	NOT PROVEN - MAY UNDERESTIMATE CAPABILITY TO SCOUR CHANNEL BY FACTOR OF 3

2

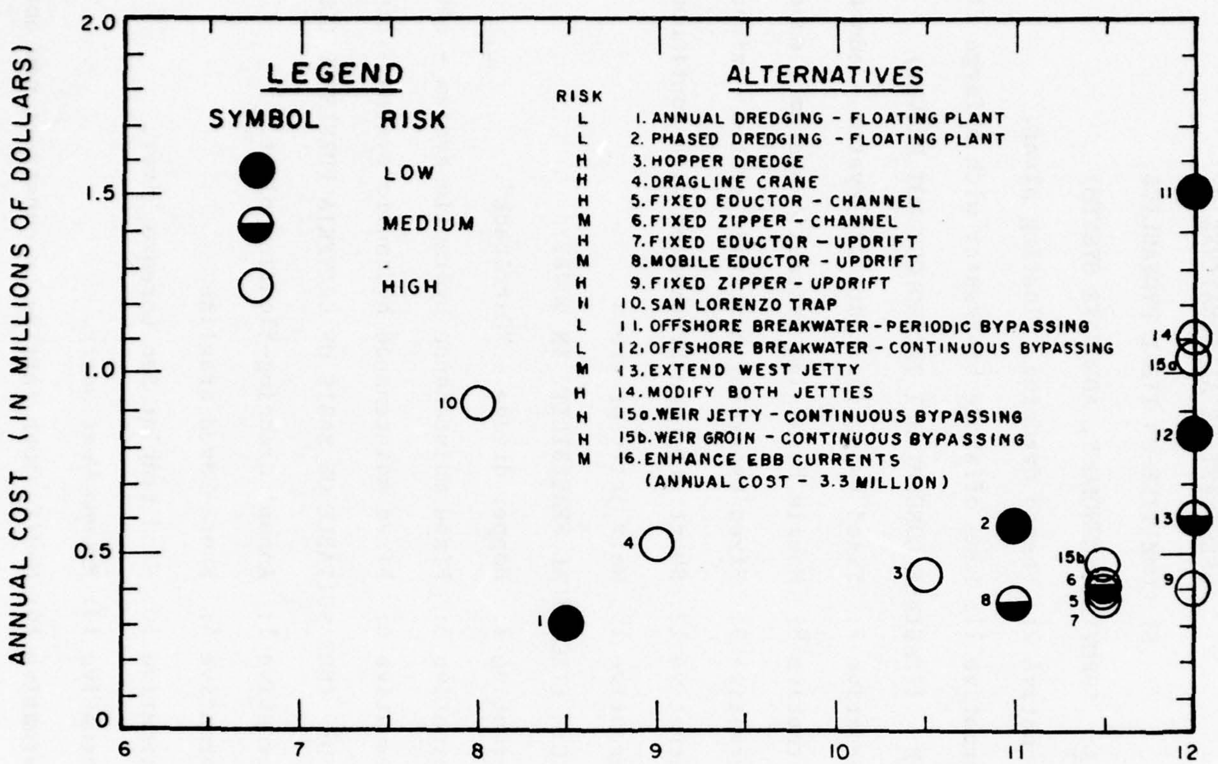

FIGURE E19: NUMBER OF MONTHS PER YEAR HARBOR REMAINS OPEN

COST EFFECTIVENESS

TABLE E23

ASSESSMENT OF ALTERNATIVES

BY COMMITTEE ON TIDAL HYDRAULICS

CATEGORY A: (PROVEN TECHNOLOGY, ADEQUATE SYSTEM)

Alternative 2: Phased dredging, floating plant.

Alternative 11: Long offshore breakwater with a large floating plant.

CATEGORY B: (FEASIBLE CONCEPT BUT TECHNOLOGY NOT PROVEN)

Alternative 7: Fixed bypassing hydraulic system - eductor.

Alternative 8: Mobile bypassing hydraulic system - eductor.

Alternative 9: Fixed bypassing hydraulic system - zipper.

Alternative 12: Short offshore breakwater with continuous bypassing.

Alternative 15: Weir jetty or groin

CATEGORY C: (TECHNICAL FEASIBILITY IN DOUBT)

Alternative 3: Hopper dredge - "Currituck".

Alternative 5: Fixed maintenance hydraulic system - eductor.

Alternative 6: Fixed maintenance hydraulic system - zipper.

CATEGORY D: (NOT SUITABLE ON BASIS OF CRITERIA FURNISHED COMMITTEE)

Alternative 1: Annual dredging-floating plant.

Alternative 4: Shore-based dragline.

Alternative 10: Sand trap at San Lorenzo River.

Alternative 13: Extend West Jetty.

Alternative 14: Modify both jetties or construct new entrance.

Alternative 16: Enhance ebb currents.

SECTION F

CONCLUSIONS AND RECOMMENDATIONS

CONTENTS

SECTION F

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

F.1 The net littoral transport rate at Santa Cruz Harbor is 300,000 to 500,000 cubic yards per year from west to east. The primary shoaling mechanism is the transport of sand around the head of the west jetty forming a tip shoal in the entrance channel. After the tip shoal forms, considerable quantities of littoral drift may bypass the harbor by natural processes. Dredging the bar that forms across the harbor entrance decreases natural bypassing and increases shoaling rates.

F.2 The first year of the two-year, multi-year interim dredging program succeeded in keeping the harbor open through most of the winter; however, the program can be improved to increase its effectiveness at less cost to the government and at less risk to the contractor. The trap capacity of the harbor entrance is too small; and, considering present limitations of dredging technology, only a major modification of the configuration of the harbor structures would prevent closure of the harbor during a severe storm or series of storms. More pay yardage was dredged from the harbor entrance during the first year of the interim two-year dredging program than in previous years. This can be attributed to several causes including a severe winter, severance of the bar that conveys natural bypassing, and diversion of more littoral drift into the entrance as a result of bar-severance. Also, the use of more appropriate methods of estimating pay quantities may have resulted in accounting for some yardage that would not have been reported under past practices.

RECOMMENDATIONS

F.3 Several monitoring programs are in effect either in the Santa Cruz area or throughout Monterey Bay. They include beach surveys, condition surveys, hydrographic surveys, aerial photography, and wave-gaging. These programs have several purposes,

including the monitoring of beach erosion, dredging effects and, in the past, the WES operation. In view of their multi-purpose nature, it is recommended that these programs be reviewed to determine whether any of them can be combined to save costs or can be conducted on a more routine basis to give more useful results. The dredging operation should be monitored and kept under constant scrutiny to develop procedures and controls that would make the system more effective.

F.4 Two methods of maintaining the harbor channel are recommended for further consideration. The first and most feasible at the present time is through continuation of the phased-dredging plan. Each contract should be limited to two to three years until more experience is gained to ensure that the program can work and that fair and reasonable contracts are consumated. The second method is construction of a long offshore breakwater and periodic dredging of the protected sand trap which it will create. More detailed studies of these solutions should be underaken. It is further recommended that model studies of all structural solutions deemed feasible in this report be conducted using a semi-moveable-bed model in order to optimize configurations and to evaluate their effectiveness.

APPENDIX 1

REFERENCES

APPENDIX 1

REFERENCES

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APPENDIX 2

ENERGY FLUX CALCULATIONS

The longshore energy flux, $P_{\ell s}$, was calculated using methods described in Shore Protection Manual (1973) for each wave condition given in the wave climate, by use of the formula:

$$P_{\ell s} = \left(\frac{\rho g H_b^2}{8} nC \cos \alpha_b\right) \sin \alpha_b \qquad (1)$$

Where α_b = angle of wave crest relative to bottom contours at breaking;

H_b = breaker height;

ρ = density of water;

g = acceleration due to gravity;

nC = group wave velocity.

The breaker height and angle of approach at breaking were determined by simultaneous solution of the following five equations which describe wave transformations. The breaker height is given by an empirical equation developed by Le Méhauté and Koh (1966):

$$\frac{H_b}{H_o} = .76S^{1/7}\left(\frac{H_o}{L_o}\right)^{-1/4} \qquad (2)$$

Where S = the bottom slope;

H_o = the deep-water wave height; and

L_o = the deep-water wave length.

This relation derived from empirical data given in Shore Protection Manual is valid in the regions where:

$$1/5 \geq S \geq 1/50$$

and

$$.09 \geq \frac{H_o}{L_o} \geq .002$$

In the data set describing the wave climate most of the wave steepnesses, H_o/L_o, and beach slopes are within these limts. The exceptions were for some of the Southern Hemisphere swell, which had steepnesses below the validity limit. In this case, the refracted and shoaled height for d = 18' was doubled and used as H_b. The equivalent deep-water wave height, H'_o, was taken into consideration by application of the refraction coefficient which assumed straight and parallel bottom contours. Hence:

$$\frac{H'_o}{H_o} = \sqrt{\frac{\cos \alpha_o}{\cos \alpha_b}} \quad (3)$$

The angle of the breaking-wave crest relative to the shoreline, α_b, is determined by:

$$\sin \alpha_b = \tanh k_b d_b \sin \alpha_o \quad (4)$$

$$\text{Where } k_b = \frac{2\pi d_b}{L_b} \; ;$$

L_b = wave length at breaking; and

d_b = breaker depth.

The wave length at breaking is:

$$L_b = L_o \tanh (k_b d_b). \quad (5)$$

The breaker depth was assumed proportional to the depth of water, or:

$$\frac{d_b}{H_b} = c \qquad (6)$$

The value of c ranges from about 1.3 to .7, depending on bottom slope and wave steepness. A value of unity was used in the calculations as being representative of the range of predominant wave conditions expected at the project site. Equations (2) through (6) were solved simultaneously for α_b, H_b, and L_b. The method described incorporates the effect of refraction from deep water to the breaker point and solves for H_b/H'_o as a function of H'_o at the breaker point. A computer program was constructed to determine the breaking-wave-energy flux for each wave condition given in the wave climate.

The preceding procedure calculated the height and angle of approach at breaking from empirical relationships. The shoaling coefficient, K_s, was not used to determine H since it underpredicts wave shoaling by as much as a factor of 2 near the breaker point for longer period waves. The method requires use of a slope, S; constant, c, relating breaker height to depth; and determination of L_b. Small variations of slope, S, have minor influences on breaker height. Because of composite slopes found in natural beaches, the bottom slope generally steepens for smaller waves. An average slope of 1 on 30 was used in the calculations. The value of c depends on the incident wave conditions; however, a value of unity appears to fit the data set and beach slope. The wave length at breaking can be 25 percent longer than given by equation (5), which is based on small amplitude theory. The effect of variation in the above variables can lead to differences of approximately ±25 percent in results of the energy-flux calculations. More accurate expressions could be employed. The slope could be given as $S = f(H_b)$, $c = f(S, H_o/L_o)$, and $L_b = f(k_d, H_b)$; however, the wave data are not accurate enough and the shoreline configuration is too variable to justify such detailed calculations.

The data base used in the energy-flux calculations was given in terms of deep-water wave statistics in the form of a three-dimensional matrix of direction, height, and period. Wave transformations were performed to obtain wave statistics at the breaker line so that the energy flux at the breaker line could be computed. Transformations were made primarily through consideration of refraction and shoaling. The effect of bottom friction and percolation on the reduction of wave heights was assumed to be of secondary importance.

Refraction diagrams were drawn using the graphical procedures described in the Shore Protection Manual. Periods ranging from 5 to 21 seconds were considered along with waves approaching from the southeast clockwise through northwest. The wave rays were brought into the 18-foot contour, where the wave angle and refraction coefficient were measured. The equivalent deep-water wave height, H_o', at the 18-foot depth was used as deep-water input to the program. Therefore, the effects of irregular bathymetry from deep water to the 18-foot depth were accounted for in this analysis.

The wave climate data are grouped by increments of deep-water H, T, and α for each month of the year. The calculation of energy flux was done by choosing representative parameters. The given data were transformed into representative heights, H_r, periods, T_r and directions. The representative value selected for calculation is defined as those values of H, T, and α that yield the same energy flux as that calculated for an even distribution of parameter values over each increment. The wave period increment is two seconds, and the average period of each increment was selected as representative, but determinations of H_r and α_r was more complex.

Equation 4-36 of SPM gives the following formula for calculating the longshore energy flux where offshore contours are straight and parallel:

$$P_{\ell s} = 18.3\ H_o^{5/2}\ (\cos\alpha_o)^{1/4}\ \sin 2\alpha_o$$

This may be written:

$$P_{\ell s} = c\ F(H_o)\ F(\alpha_o)$$

Where $c = 18.3$;

$F(H_o) = H_o^{5/2}$; and

$F(\alpha_o) = \cos^{1/4}\alpha_o\ \sin 2\alpha_o$; or

$F(\alpha_o) = 2\cos^{5/4}\alpha_o\ \sin\alpha_o$.

Given a range of H_o and α_o, representative values H_{or} and α_{or} must be selected. These values are chosen assuming H_o and α_o are evenly distributed over their incremental ranges and are bounded by upper and lower bounds denoted by subscripts "u" and "ℓ", respectively. $F(H_{or})$ is then determined by the following integral:

$$F(H_{or}) = \frac{1}{\Delta H_o} \int_{H_{o\ell}}^{H_{ou}} H_o^{5/2}\ dH_o.$$

The solution is:

$$F(H_{or}) = \frac{2}{7\Delta H_o}\ (H_{ou}^{7/2} - H_{o\ell}^{7/2}).$$

The representative H_{or} is then:

$$H_{or} = \left[\frac{2}{7\Delta H_o}\ (H_{ou}^{7/2} - H_{o\ell}^{7/2})\right]^{2/5}$$

Table 1 lists the representative wave heights for given incremental ranges:

Table 1. REPRESENTATIVE WAVE HEIGHT

Southern Hemisphere Swell

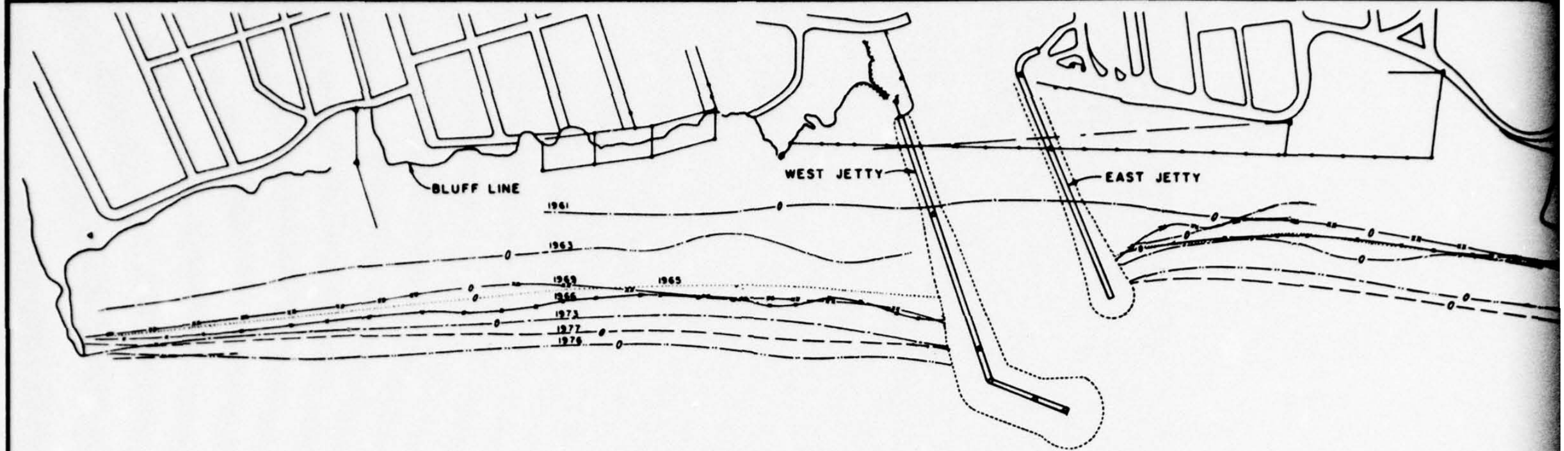
Ht. Range (ft)	Representative Ht. (ft)
.1-.9	.57
1-1.9	1.48
2-2.9	2.47
3-3.9	3.46

Northern Hemisphere Swell & Seas

Ht. Range (ft)	Representative Ht. (ft)
1-2.9	2.06
3-4.9	4.01
5-6.9	5.99
7-8.9	7.98
9-10.9	9.97
11-12.9	11.97
13-14.9	13.97
15-16.9	15.96
17-18.9	17.96
19-20.9	19.96
21-22.9	21.96
23-24.9	23.96
25-26.9	25.96
27+	27.96

Because of the irregular bathymetry off the Santa Cruz coast, α_o could not be used directly in the deep-water energy-flux formula. Instead the refracted direction at the 18-foot contour was selected as the representative wave direction.

LIST OF PLATES



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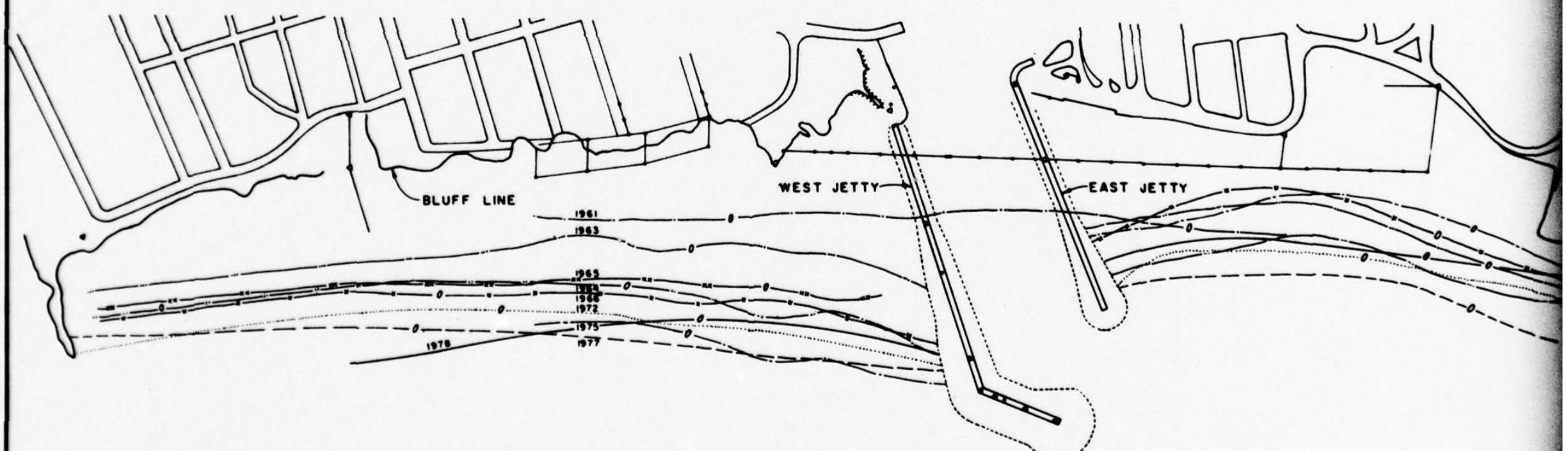

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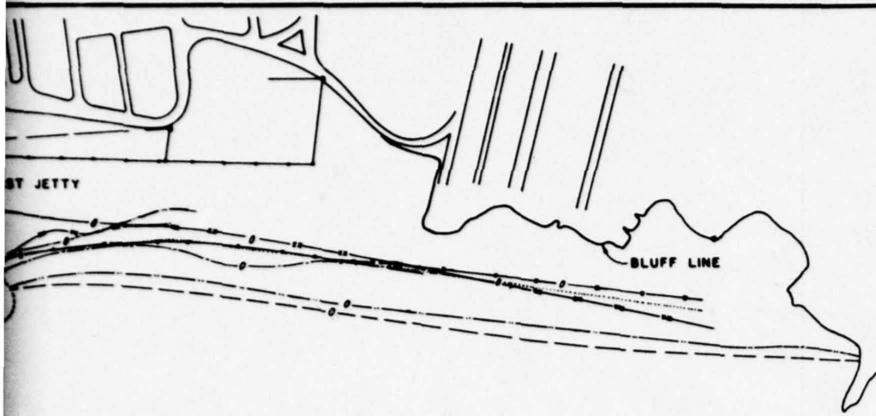


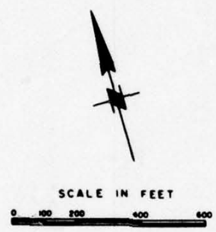
LEGEND

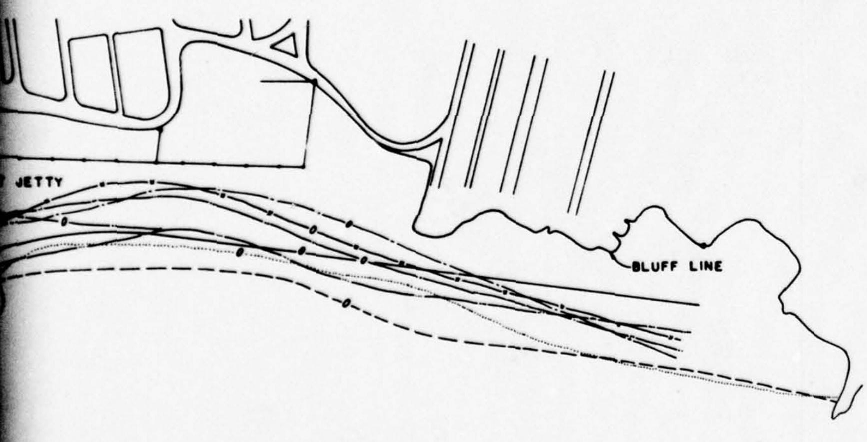
DECEMBER 1961	
SEPTEMBER 1963	
AUGUST 1965	
AUGUST 1966	
*JULY 1969	
*SEPTEMBER 1973	
AUGUST 1976	
AUGUST 1977	

NOTES

1. ELEVATIONS AND SOUNDINGS ARE BASED ON THE DATUM OF MEAN LOWER LOW WATER.
2. SHORELINE CONTOURS BASED ON CORPS OF ENGINEERS BEACH EROSION SURVEYS.

TOURS





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MAY 1963	
FEBRUARY 1964	
MARCH 1965	
MARCH 1966	
*DECEMBER 1972	
MARCH 1975	
APRIL 1977	
FEBRUARY 1978	

*SHORELINE DRAWN FROM AERIAL PHOTOGRAPHS.

TOURS

SANTA CRUZ HARBOR, CALIFORNIA
SHOALING STUDY

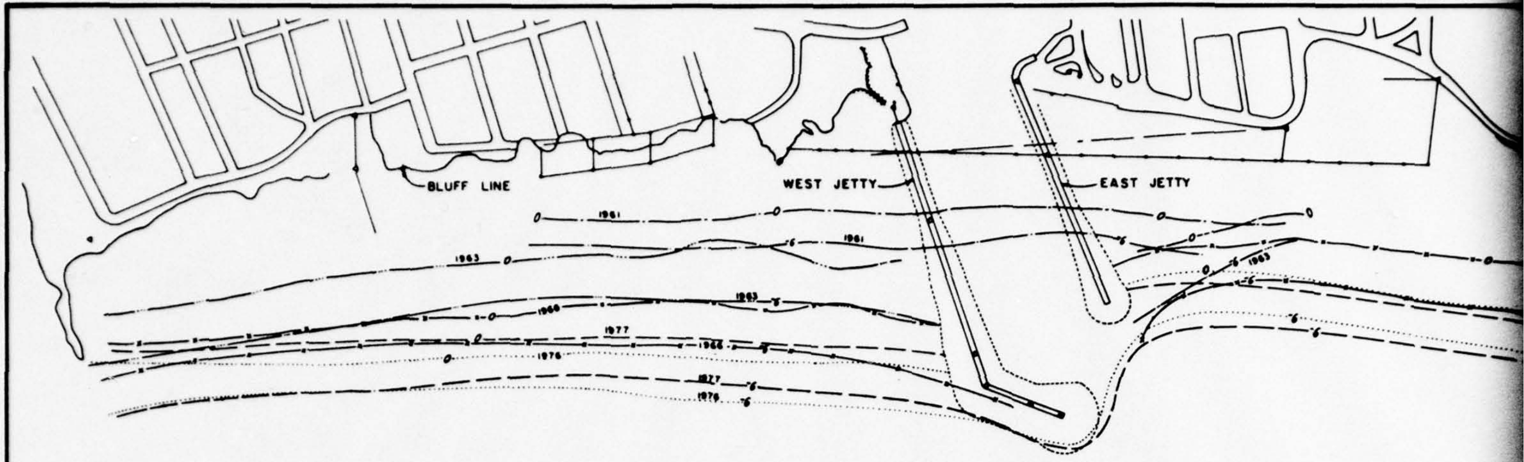
SHORELINE CHANGES

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO

Prepared & Submitted By: Moffatt & Nichol, Engineers Long Beach, California	To Accompany Shoaling Study Report Dated : June 1978

PLATE I

2



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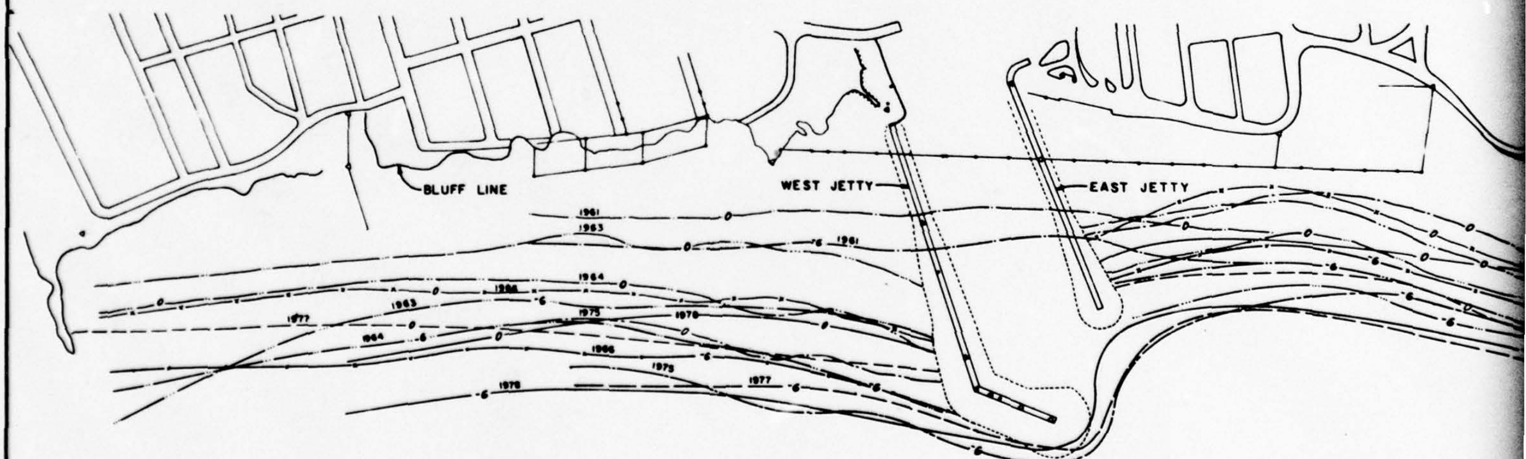

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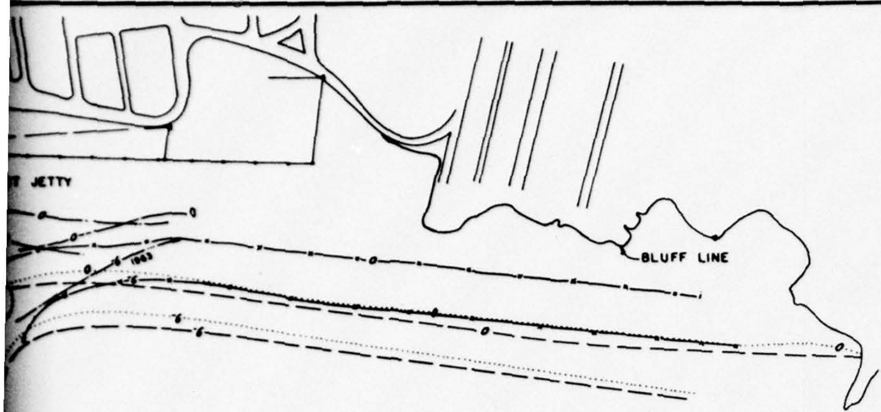

LEGEND

- DECEMBER 1961
- SEPTEMBER 1963
- AUGUST 1966
- AUGUST 1976
- AUGUST 1977

NOTES

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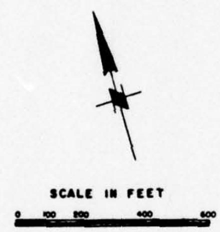
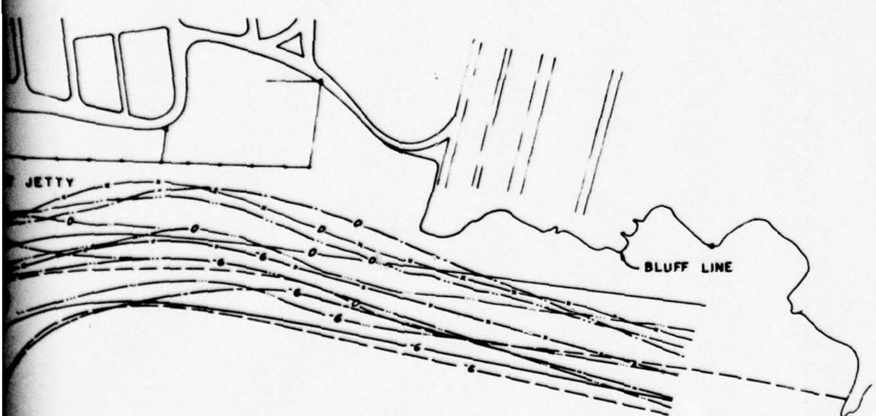
TOURS





LEGEND

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- MAY 1963
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- MARCH 1966
- MARCH 1975
- APRIL 1977
- FEBRUARY 1978

TOURS

SANTA CRUZ HARBOR, CALIFORNIA
SHOALING STUDY

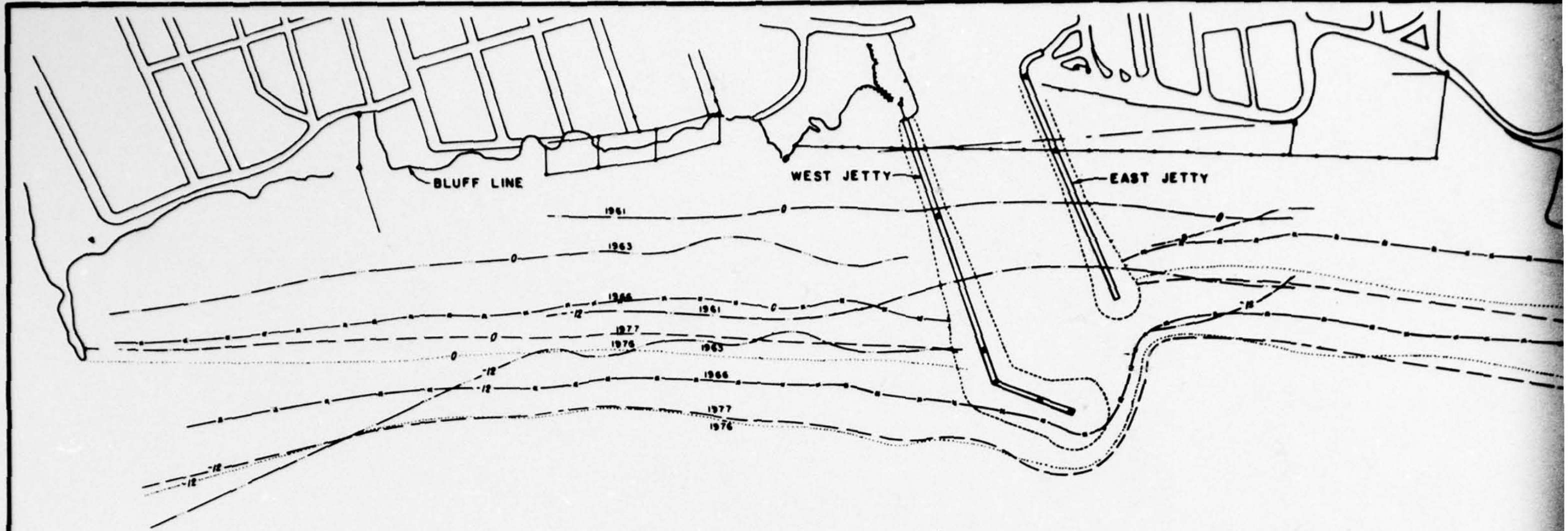
SHORELINE AND CONTOUR CHANGES -6 FT.

U. S. ARMY ENGINEER DISTRICT, SAN FRANCISCO

Prepared & Submitted By: Moffatt & Nichol, Engineers Long Beach, California	To Accompany Shoaling Study Report Dated: June 1978

2

PLATE 2



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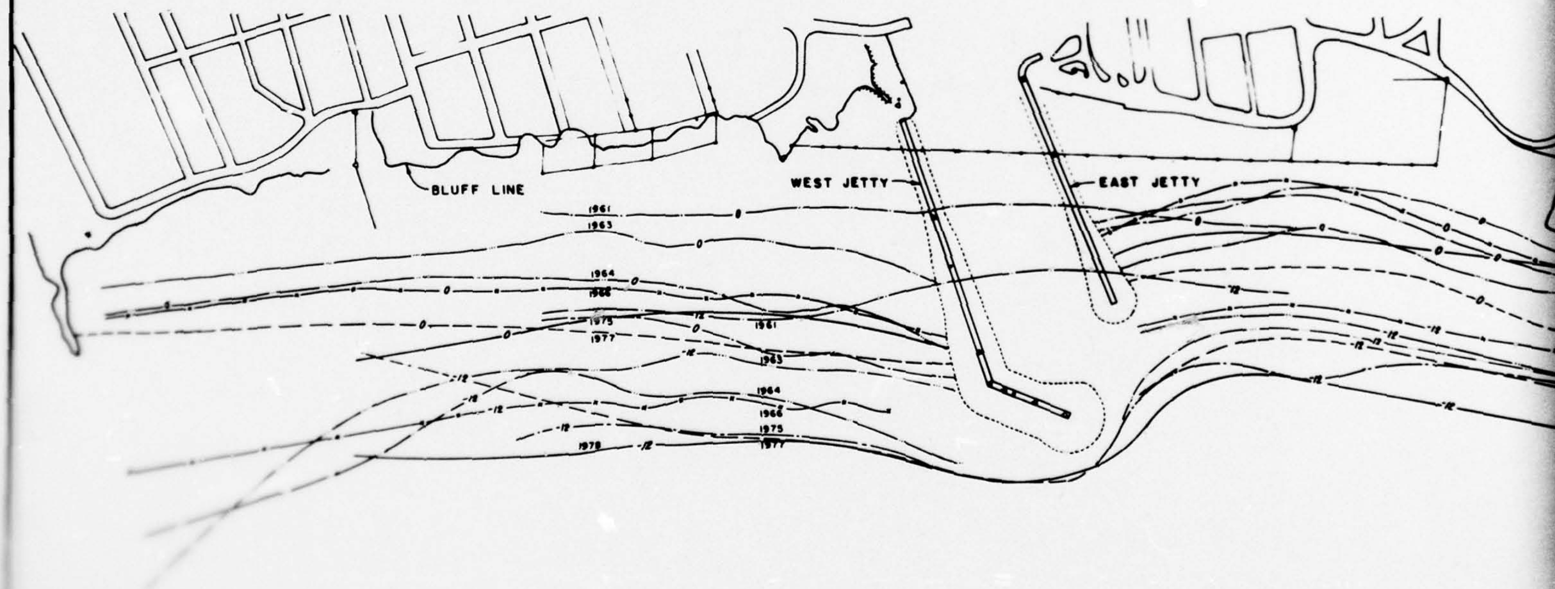

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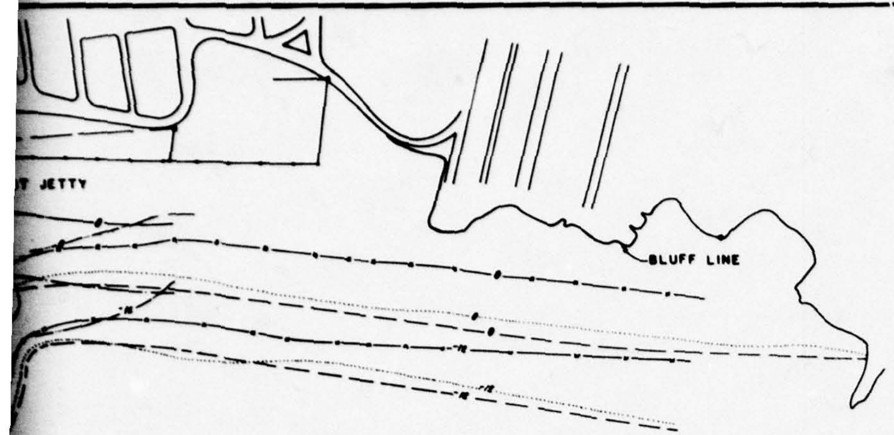

LEGEND

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AUGUST	1966
AUGUST	1976
AUGUST	1977

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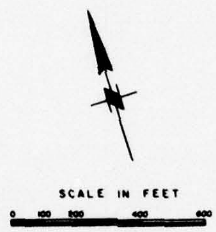
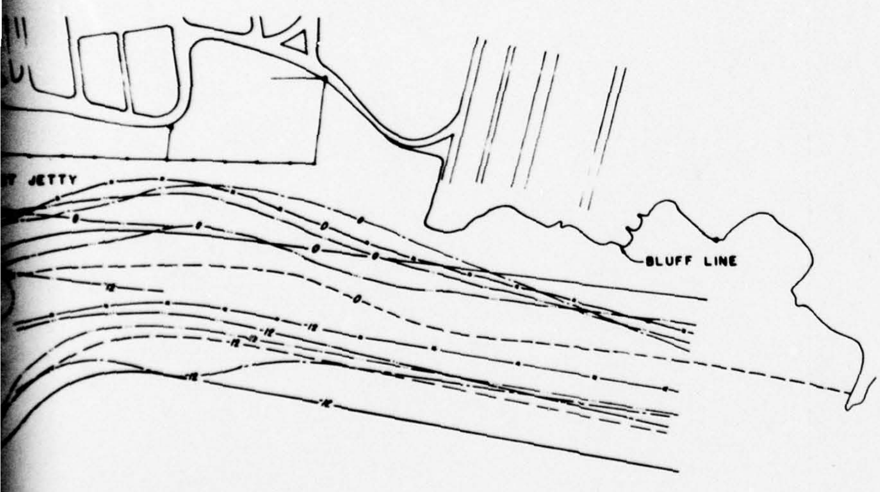
TOURS





LEGEND

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MAY	1963
FEBRUARY	1964
MARCH	1966
MARCH	1975
APRIL	1977
FEBRUARY	1978

TOURS

SANTA CRUZ HARBOR, CALIFORNIA
SHOALING STUDY

SHORELINE AND CONTOUR CHANGES - 12 FT.

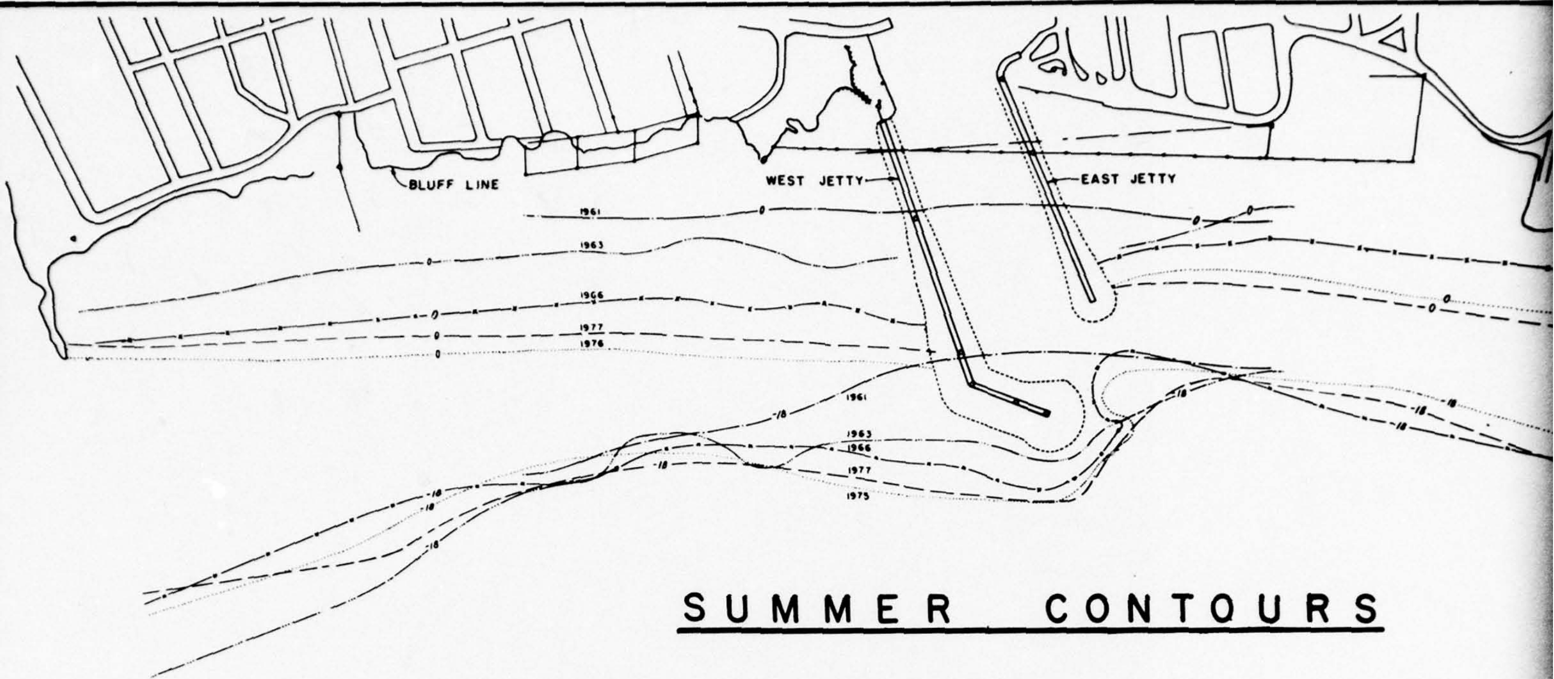
U. S. ARMY ENGINEER DISTRICT, SAN FRANCISCO

Prepared & Submitted By:
Moffatt & Nichol, Engineers
Long Beach, California

To Accompany
Shoaling Study Report
Dated : June 1978

PLATE 3

2



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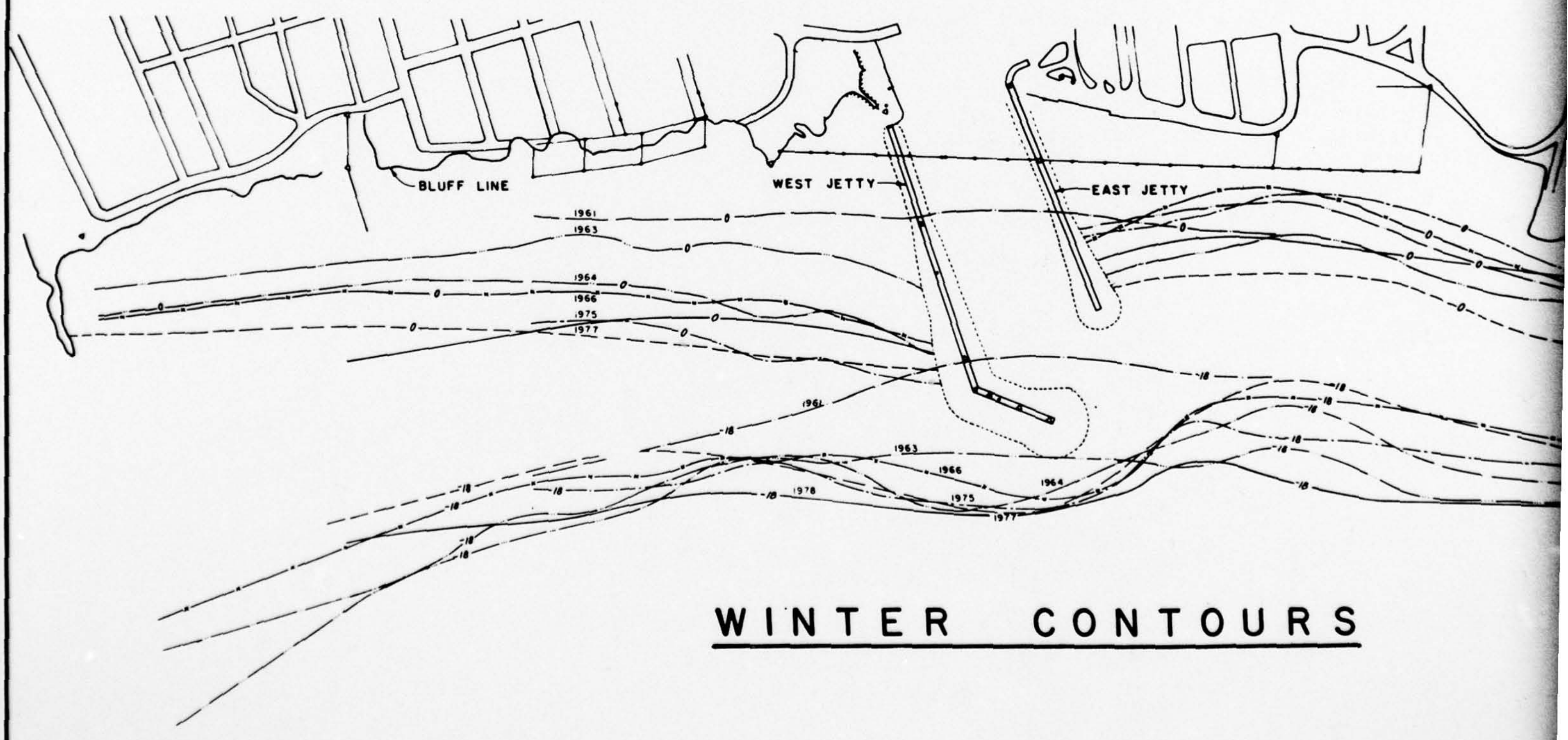

WINTER CONTOURS

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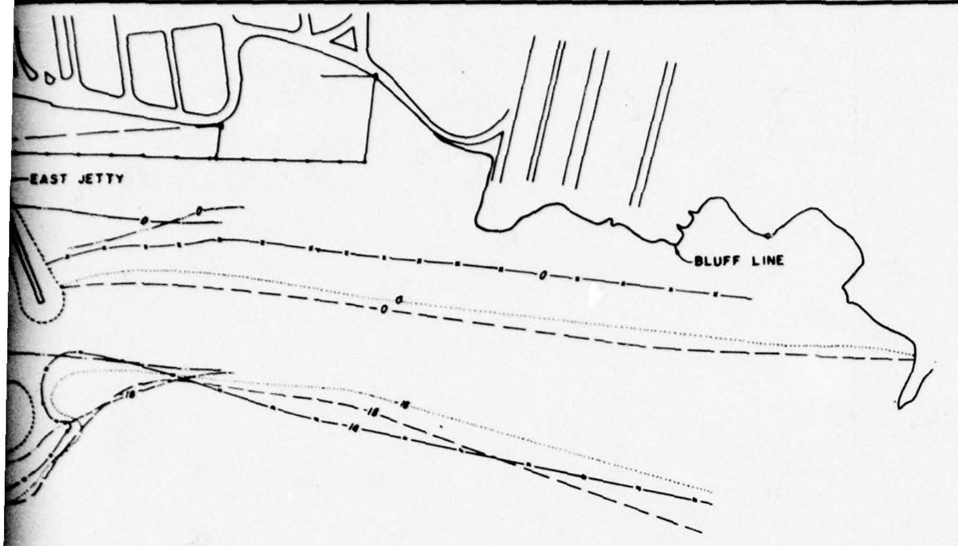

LEGEND

DECEMBER 1961	
SEPTEMBER 1963	
AUGUST 1966	
AUGUST 1976	
AUGUST 1977	

NOTES

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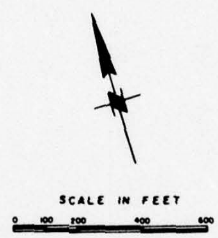
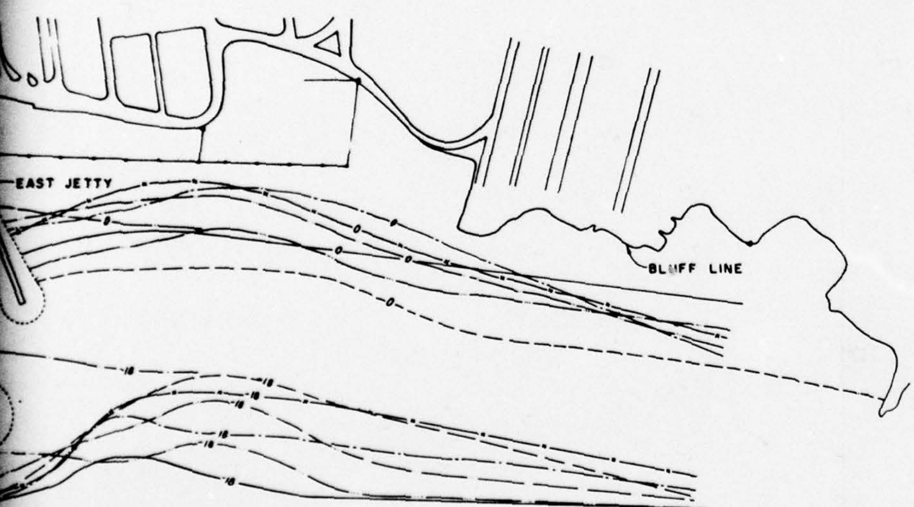
CONTOURS





LEGEND

DECEMBER 1961	
MAY 1963	
FEBRUARY 1964	
MARCH 1966	
MARCH 1975	
APRIL 1977	
FEBRUARY 1978	

ONTOURS

SANTA CRUZ HARBOR, CALIFORNIA
SHOALING STUDY

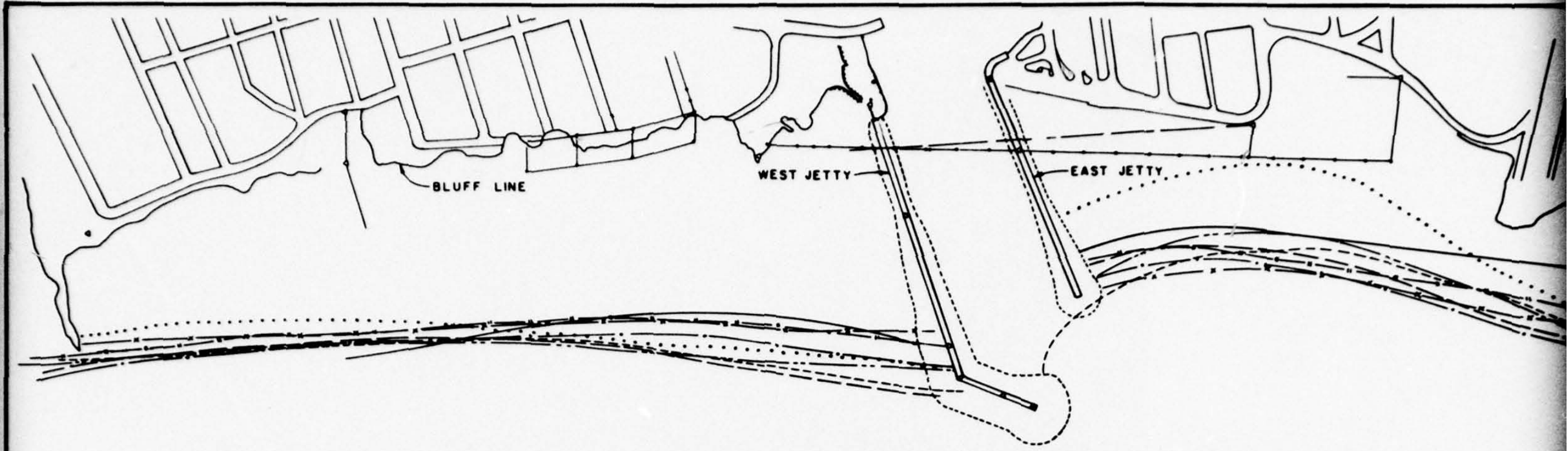
SHORELINE AND CONTOUR CHANGES - 18 FT.

U. S. ARMY ENGINEER DISTRICT, SAN FRANCISCO

Prepared & Submitted By: Moffatt & Nichol, Engineers Long Beach, California	To Accompany Shoaling Study Report Dated: June 1978

PLATE 4

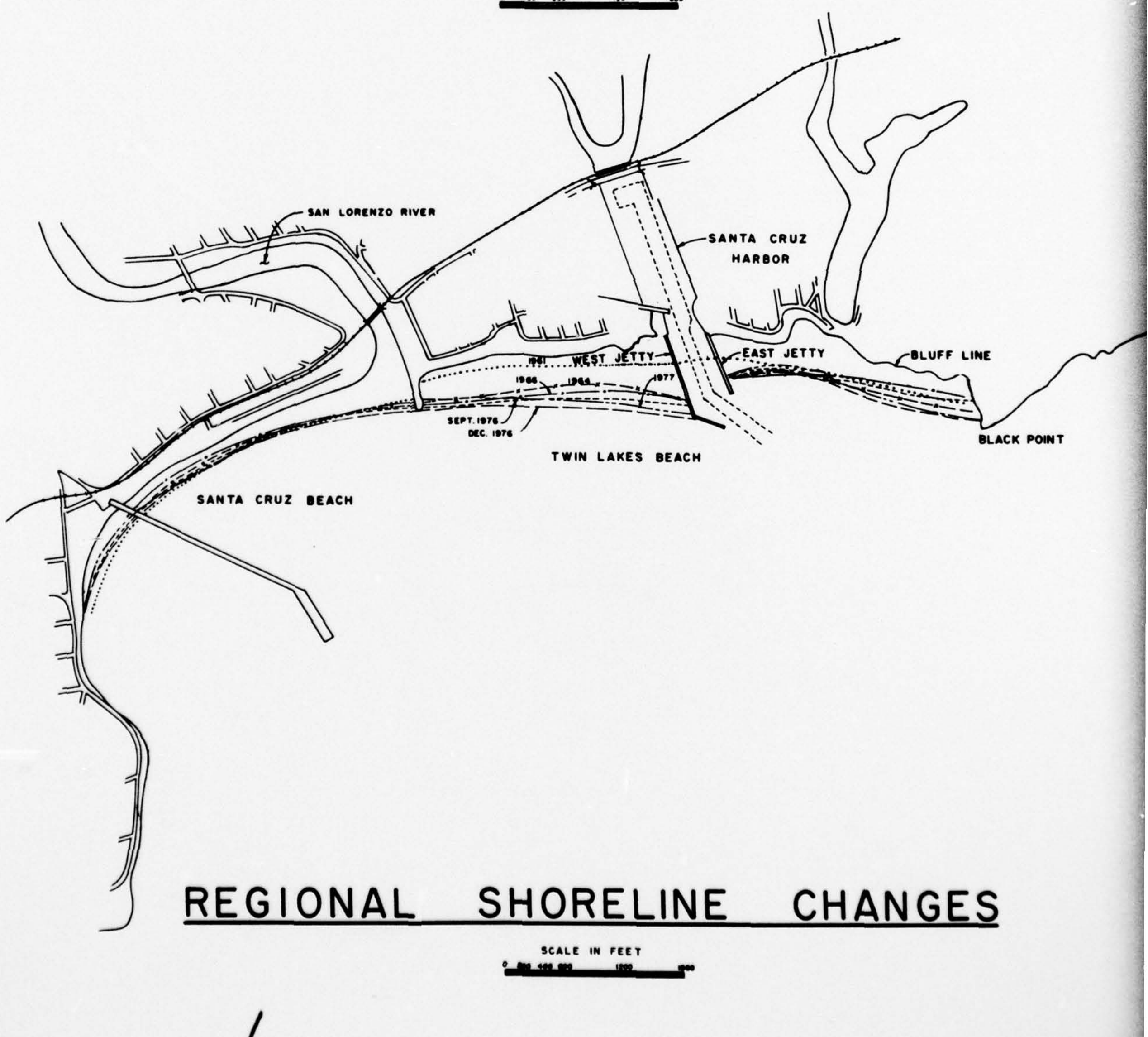
2



SEASONAL SHORELINE CHANGES

SCALE IN FEET

0 100 200 400

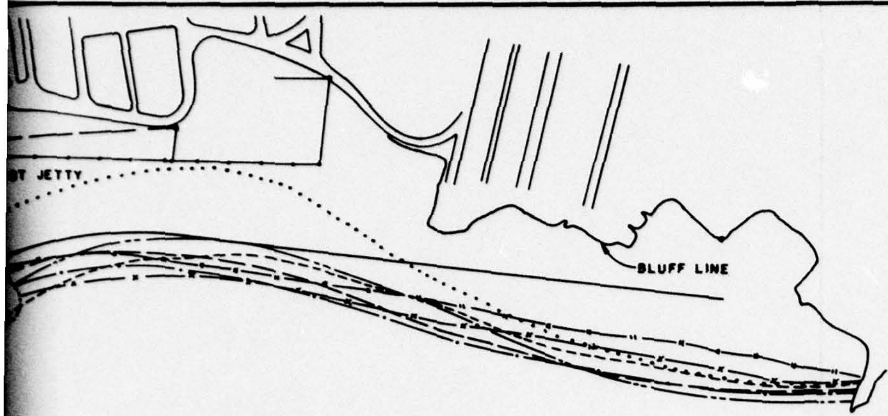


REGIONAL SHORELINE CHANGES

SCALE IN FEET

0 1200

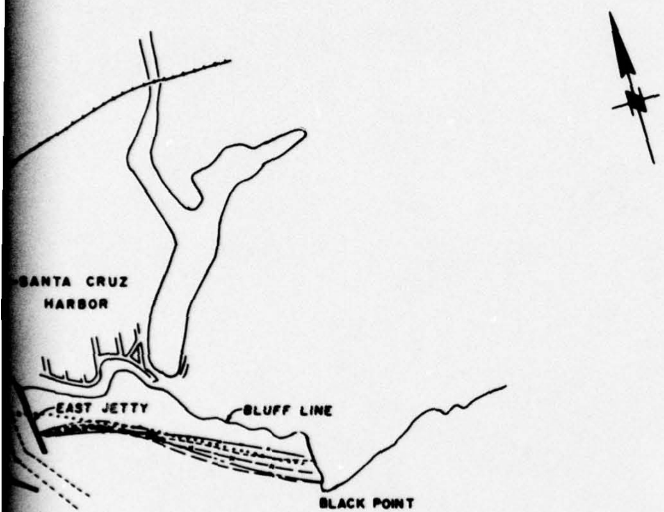
1



LEGEND

SEPTEMBER	1976	
NOVEMBER	1976	
DECEMBER	1976	
JANUARY	1977	
FEBRUARY	1977	
MARCH	1977	
AUGUST	1977	
FEBRUARY	1978	

E CHANGES



LEGEND

DECEMBER	1961*	
OCTOBER	1964	
AUGUST	1966	
SEPTEMBER	1976	
DECEMBER	1976	
APRIL	1977	

NOTES

1. ELEVATIONS AND SOUNDINGS ARE BASED ON THE DATUM OF MEAN LOWER LOW WATER.

*2. SHORELINE DRAWN FROM AERIAL PHOTOGRAPHS.

E CHANGES

SANTA CRUZ HARBOR, CALIFORNIA
SHOALING STUDY

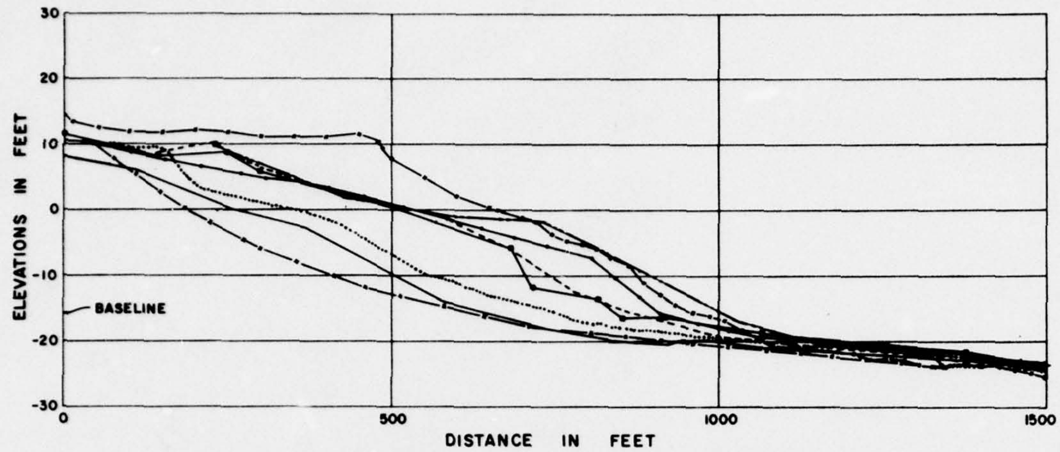
SEASONAL AND REGIONAL SHORELINE CHANGES

U. S. ARMY ENGINEER DISTRICT, SAN FRANCISCO

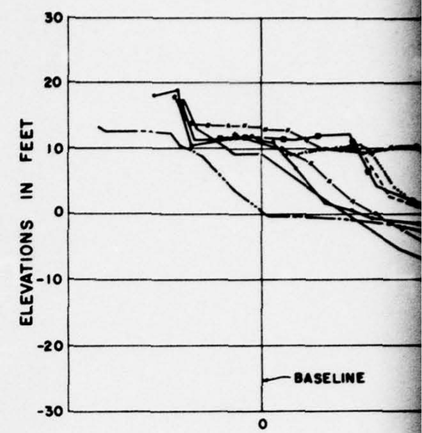
Prepared & Submitted By : Moffatt & Nichol, Engineers Long Beach, California	To Accompany Shoaling Study Report Dated : June 1978

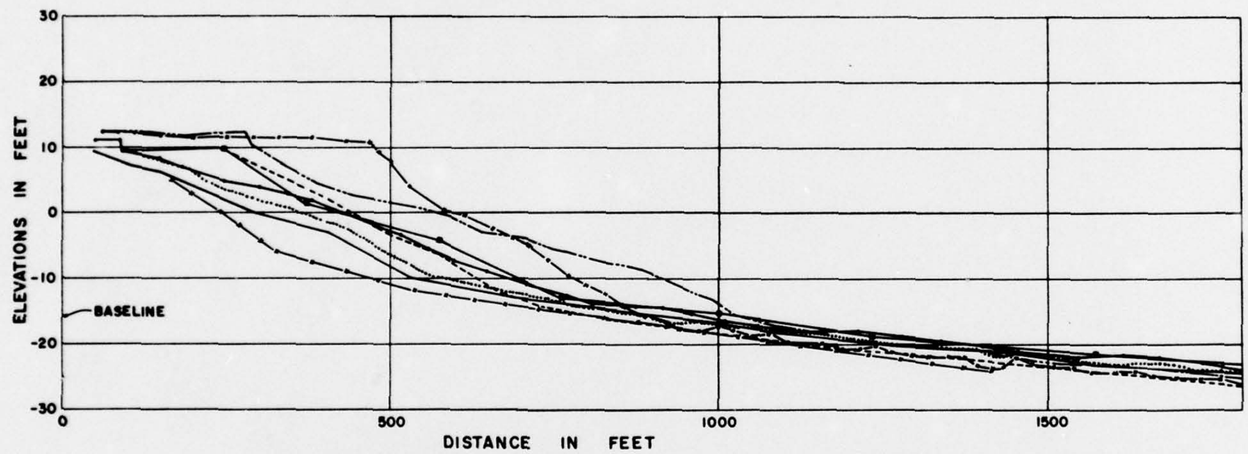
PLATE 5

2

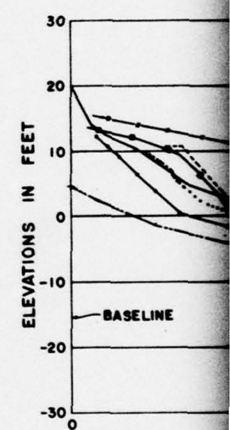


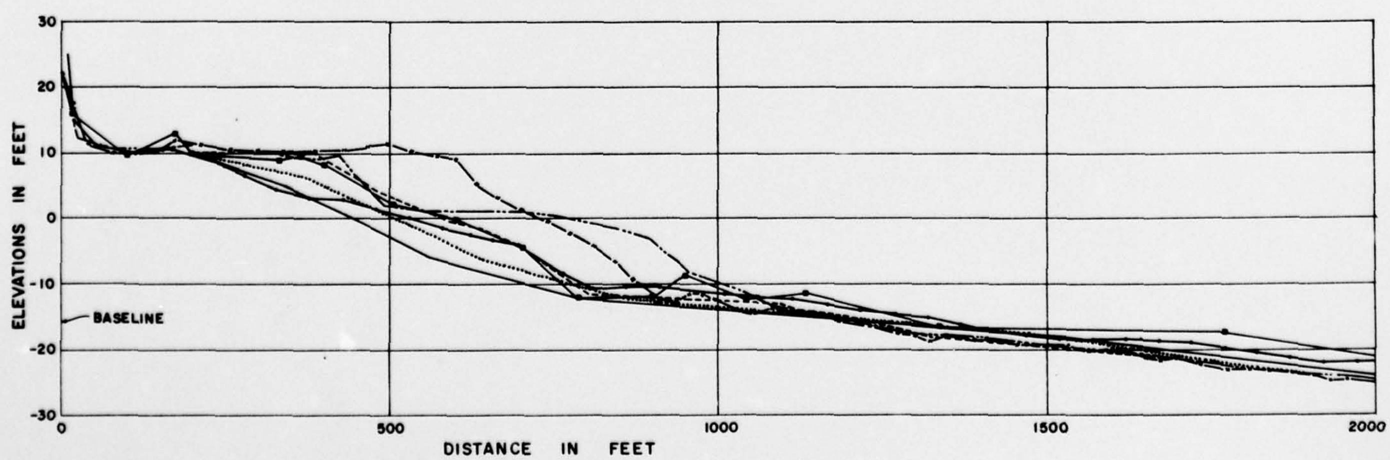
RANGE 2+00 W OLD



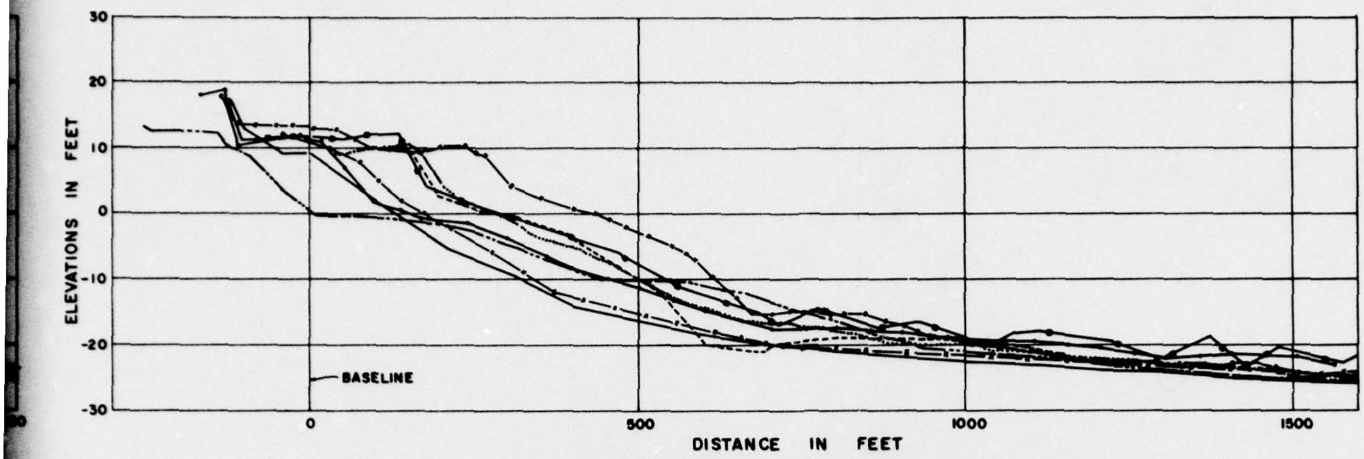


RANGE B

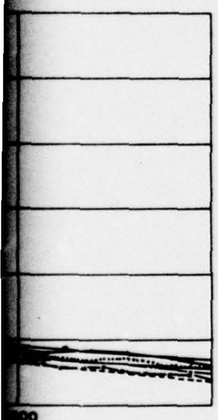


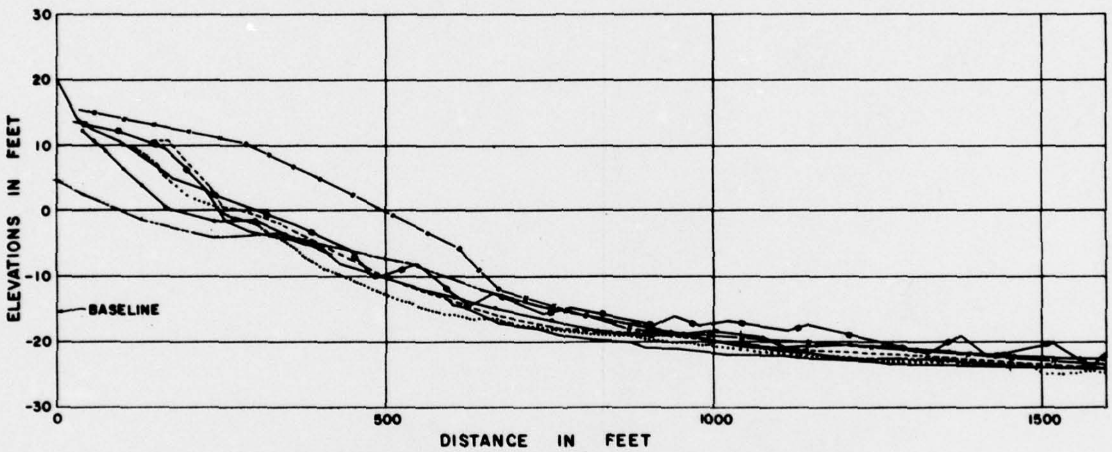


RANGE 13A

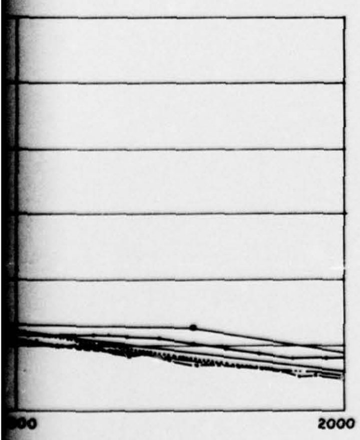


RANGE 6+00 E





RANGE 15



LEGEND

DECEMBER 1961
OCTOBER 1962
SEPTEMBER 1963
FEBRUARY 1964
OCTOBER 1964
OCTOBER 1965
AUGUST 1977
FEBRUARY 1978

NOTE

1. RANGE LINES ARE SHOWN ON PLATE 9.

SANTA CRUZ HARBOR, CALIFORNIA
SHOALING STUDY

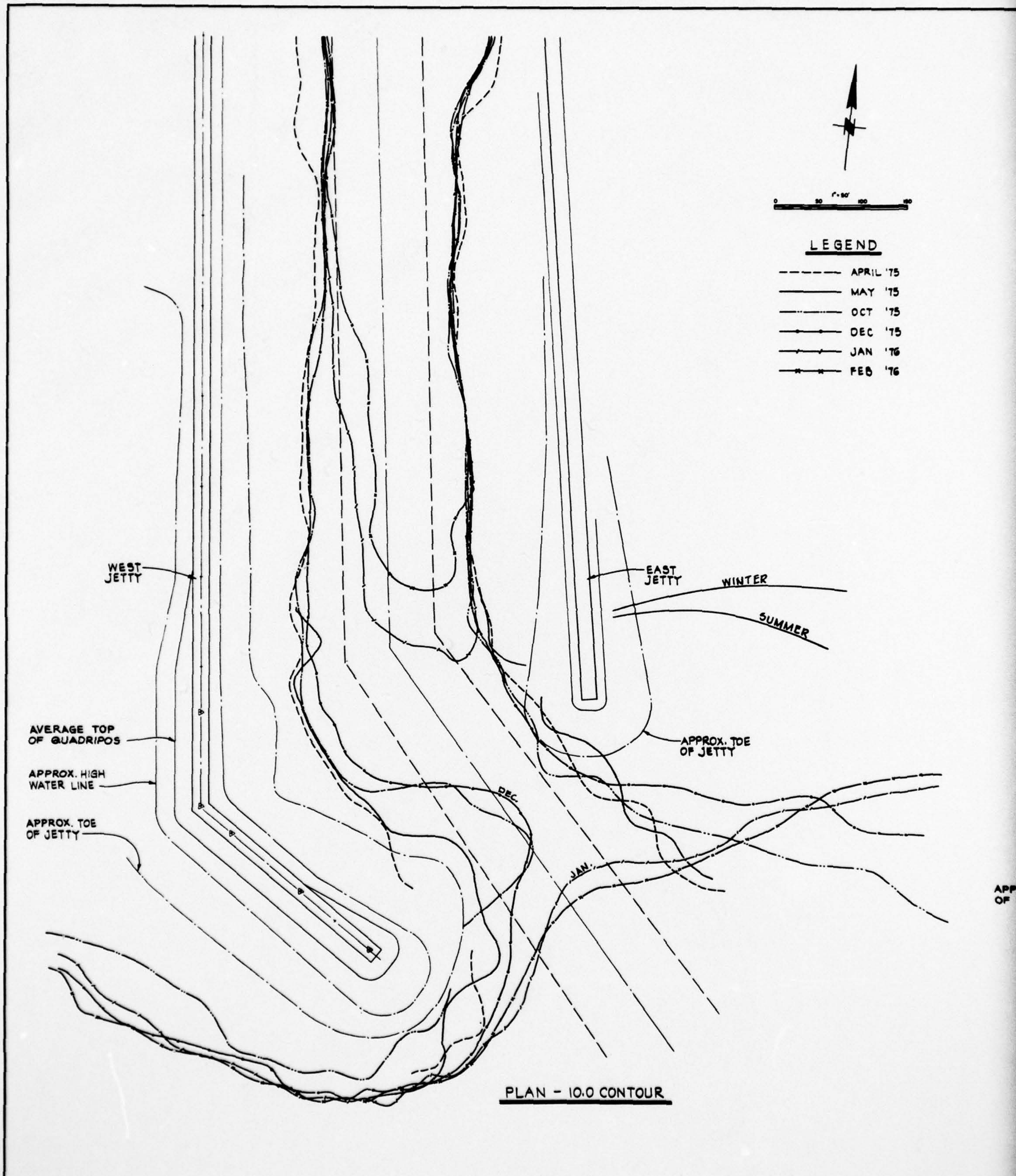
SELECTED PROFILES

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO

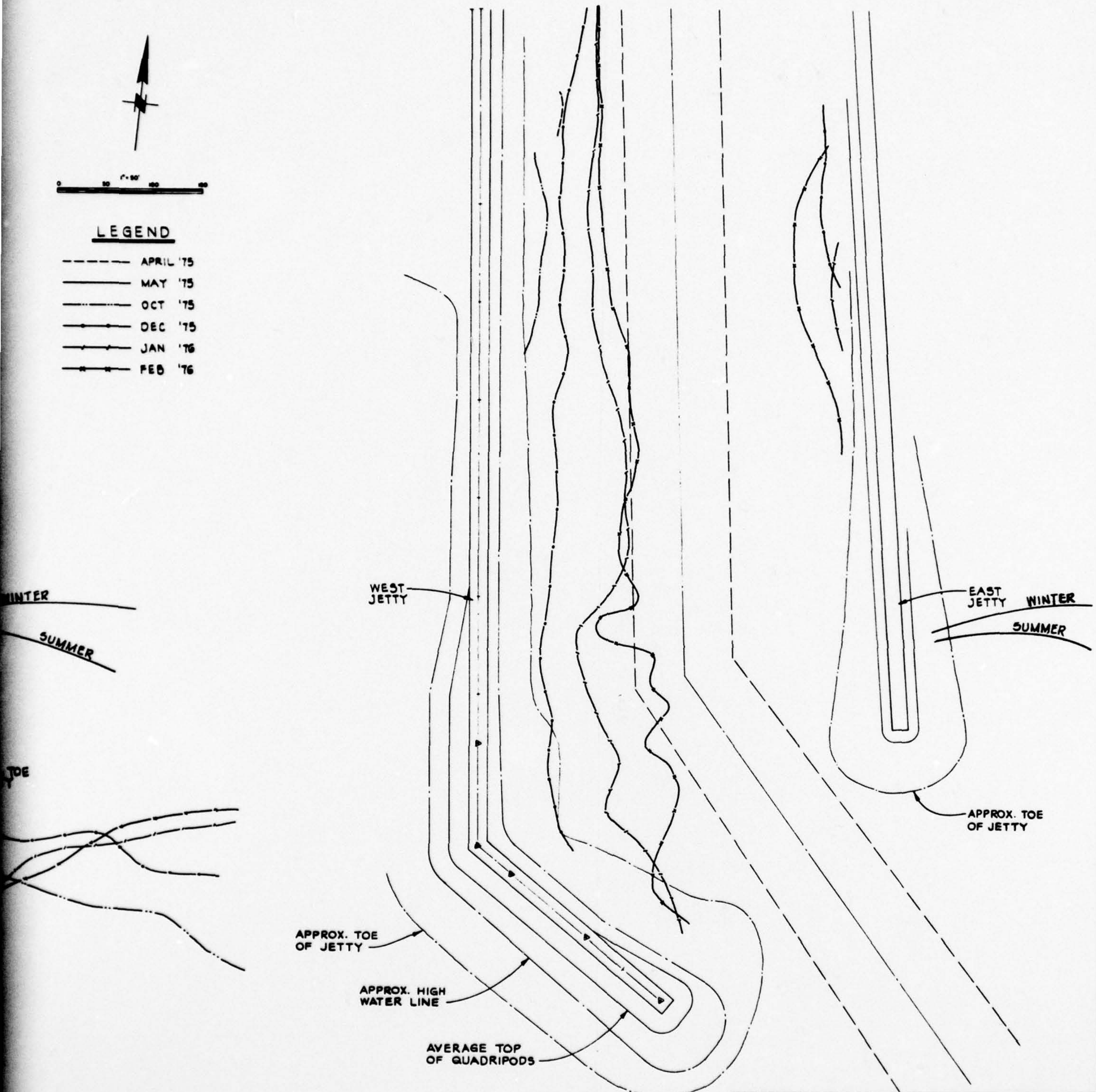
Prepared & Submitted By : Moffatt & Nichol, Engineers Long Beach, California	To Accompany Shoaling Study Report Dated : June 1978

PLATE 6

2



PLAN - 10.0 CONTOUR



PLAN - 0.0 CONTOUR

SANTA CRUZ HARBOR, CALIFORNIA

SHOALING STUDY

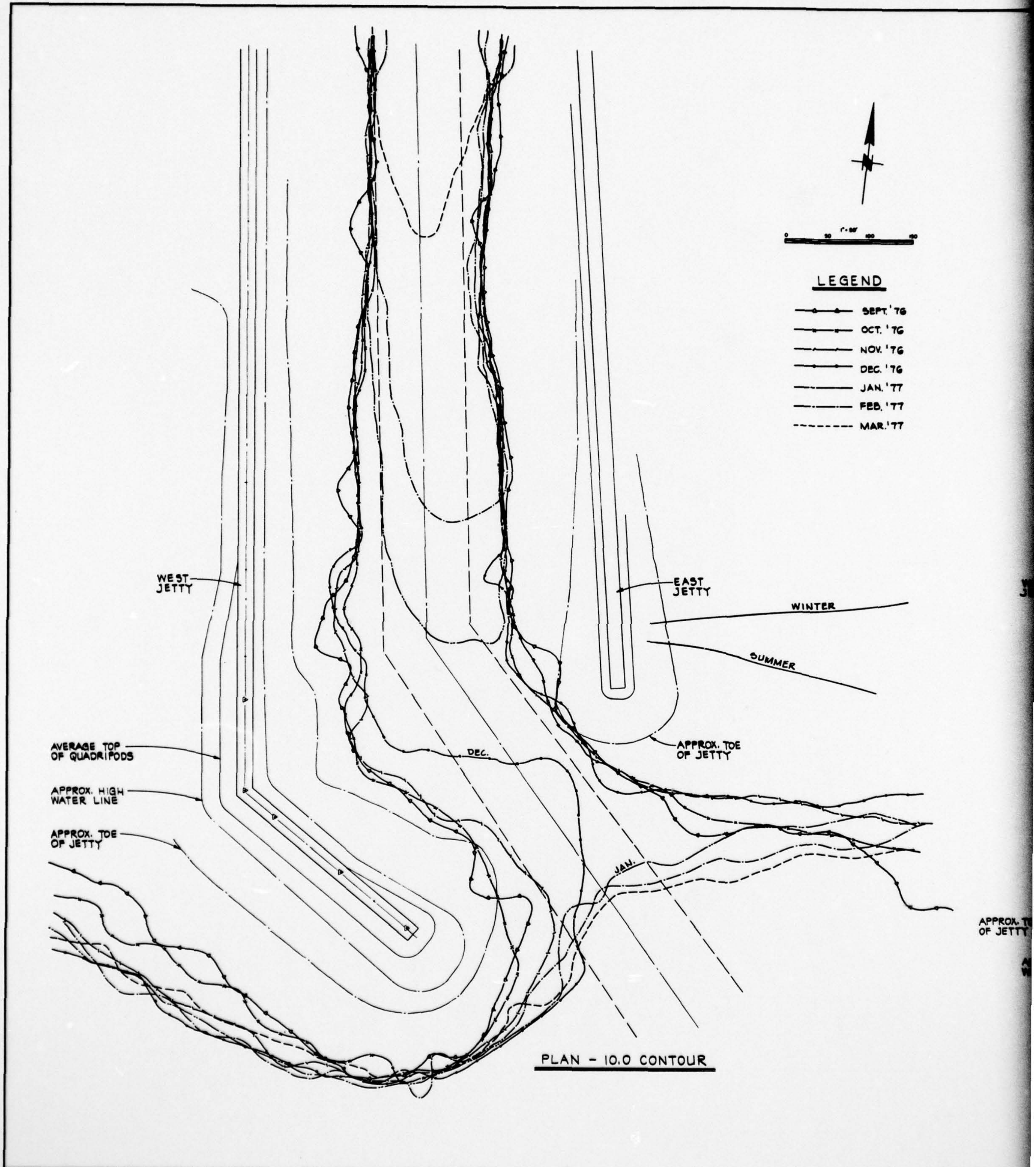
DEPTH CHANGES 1975 - 1976

U. S. ARMY ENGINEER DISTRICT, SAN FRANCISCO

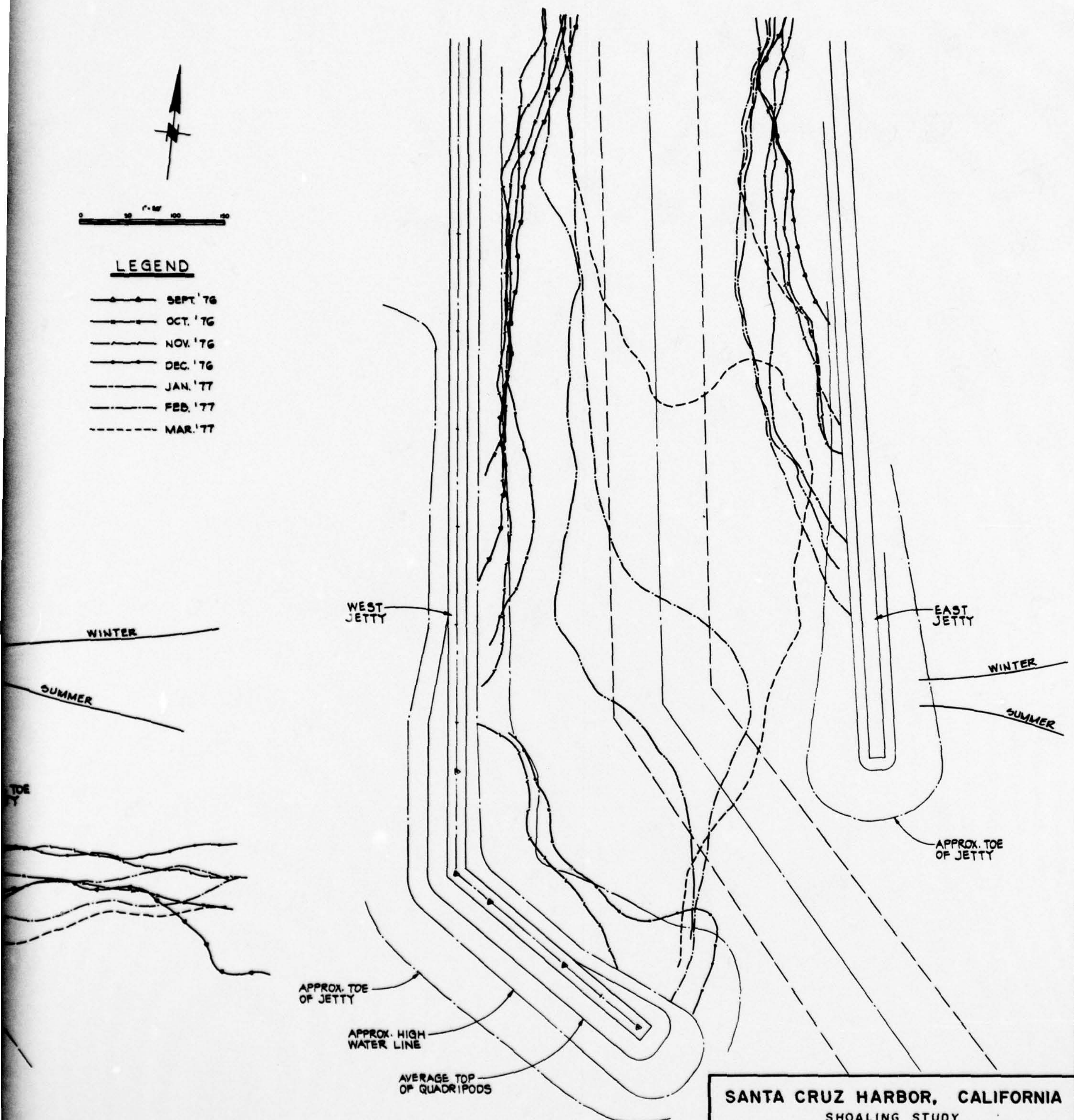
Prepared & Submitted By: Moffatt & Nichol, Engineers Long Beach, California	To Accompany Shoaling Study Report Dated: June 1978

2

PLATE 7



PLAN - 10.0 CONTOUR



PLAN - 0.0 CONTOUR

SANTA CRUZ HARBOR, CALIFORNIA

SHOALING STUDY

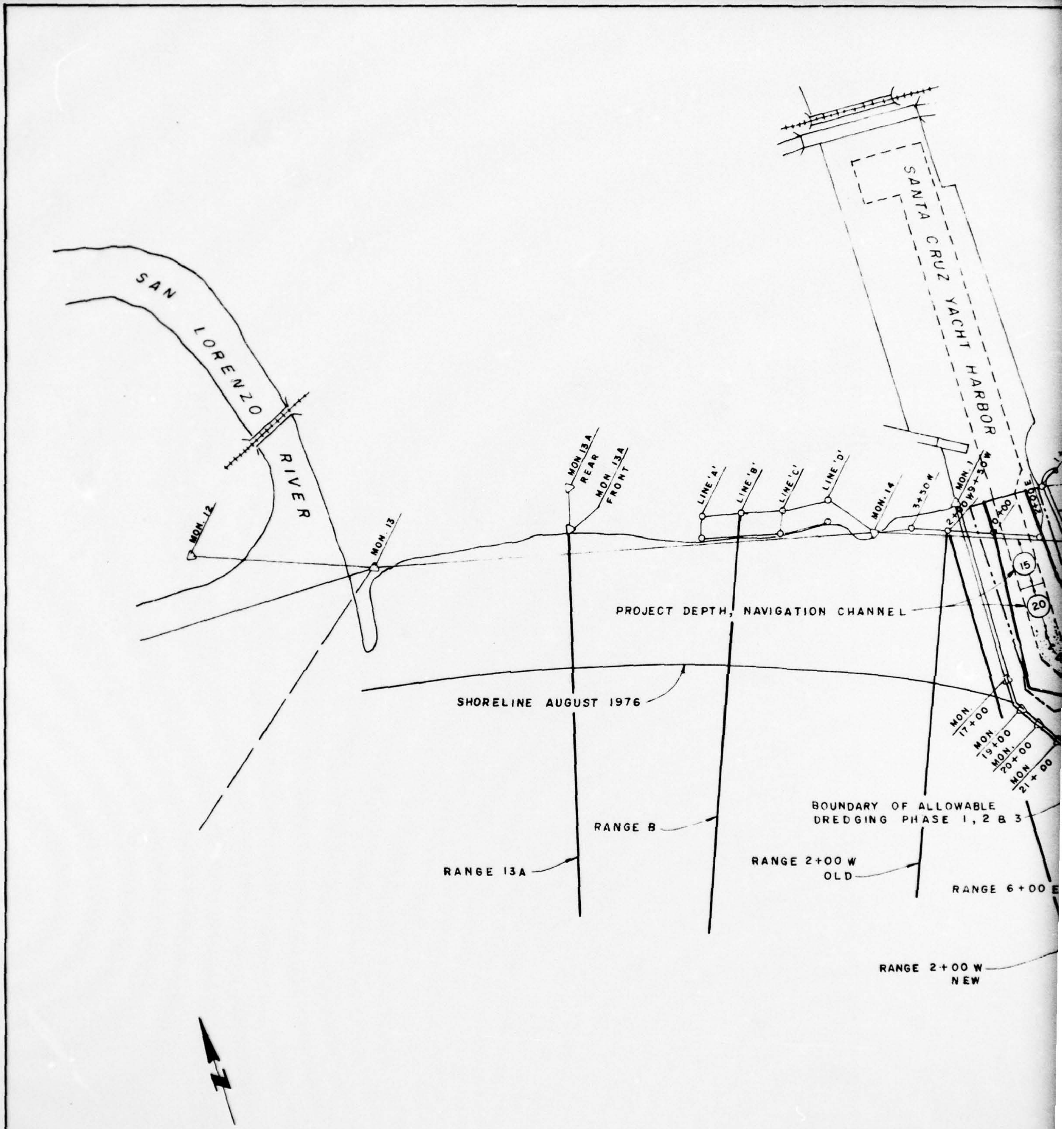
DEPTH CHANGES 1976 – 1977

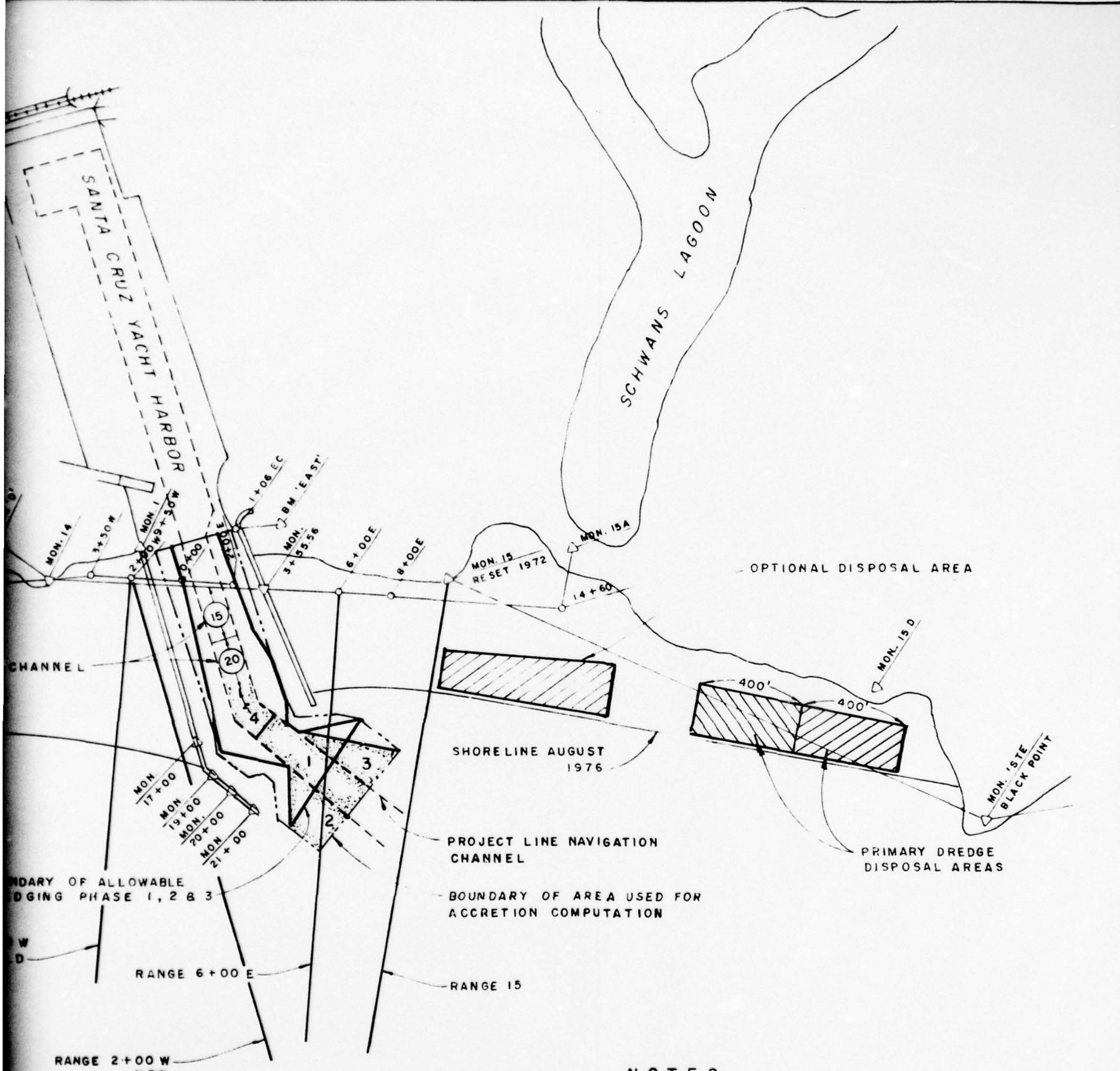
U. S. ARMY ENGINEER DISTRICT, SAN FRANCISCO

Prepared & Submitted By: Moffatt & Nichol, Engineers Long Beach, California	*To Accompany* Shoaling Study Report Dated: June 1978

PLATE 8

2


SANTA CRUZ YACHT HARBOR
SAN LORENZO RIVER
MON. 12
MON. 13
MON 13A REAR
MON 13A FRONT
LINE 'A'
LINE 'B'
LINE 'C'
LINE 'D'
MON. 14
3+50W
MON. 1
2+00W
9+50W
15
20
PROJECT DEPTH, NAVIGATION CHANNEL
SHORELINE AUGUST 1976
MON. 17+00
MON. 19+00
MON. 20+00
MON. 21+00
BOUNDARY OF ALLOWABLE DREDGING PHASE 1, 2 & 3
RANGE B
RANGE 13A
RANGE 2+00 W OLD
RANGE 6+00 E
RANGE 2+00 W NEW
SCALE IN FEET
0
500
1000



NOTES

1. SHORELINE DEFINED AS THE 0' MLLW CONTOUR

MOFFATT & NICHOL, ENGINEERS LONG BEACH, CALIFORNIA WILLIAMS HUDSON HERRON	SANTA CRUZ HARBOR SANTA CRUZ, CALIF. SHOALING STUDY DREDGING BOUNDARIES AND DISPOSAL AREAS	6 78 L-1735

2

PLATE 9

AD-A060 205 MOFFATT AND NICHOL INC LONG BEACH CA F/G 13/13
SANTA CRUZ HARBOR SHOALING STUDY, SANTA CRUZ HARBOR, CALIFORNIA--ETC(U)
JUN 78 DACW07-77-C-0023

UNCLASSIFIED NL

4 OF 4
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A060 205

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DATE
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12-78
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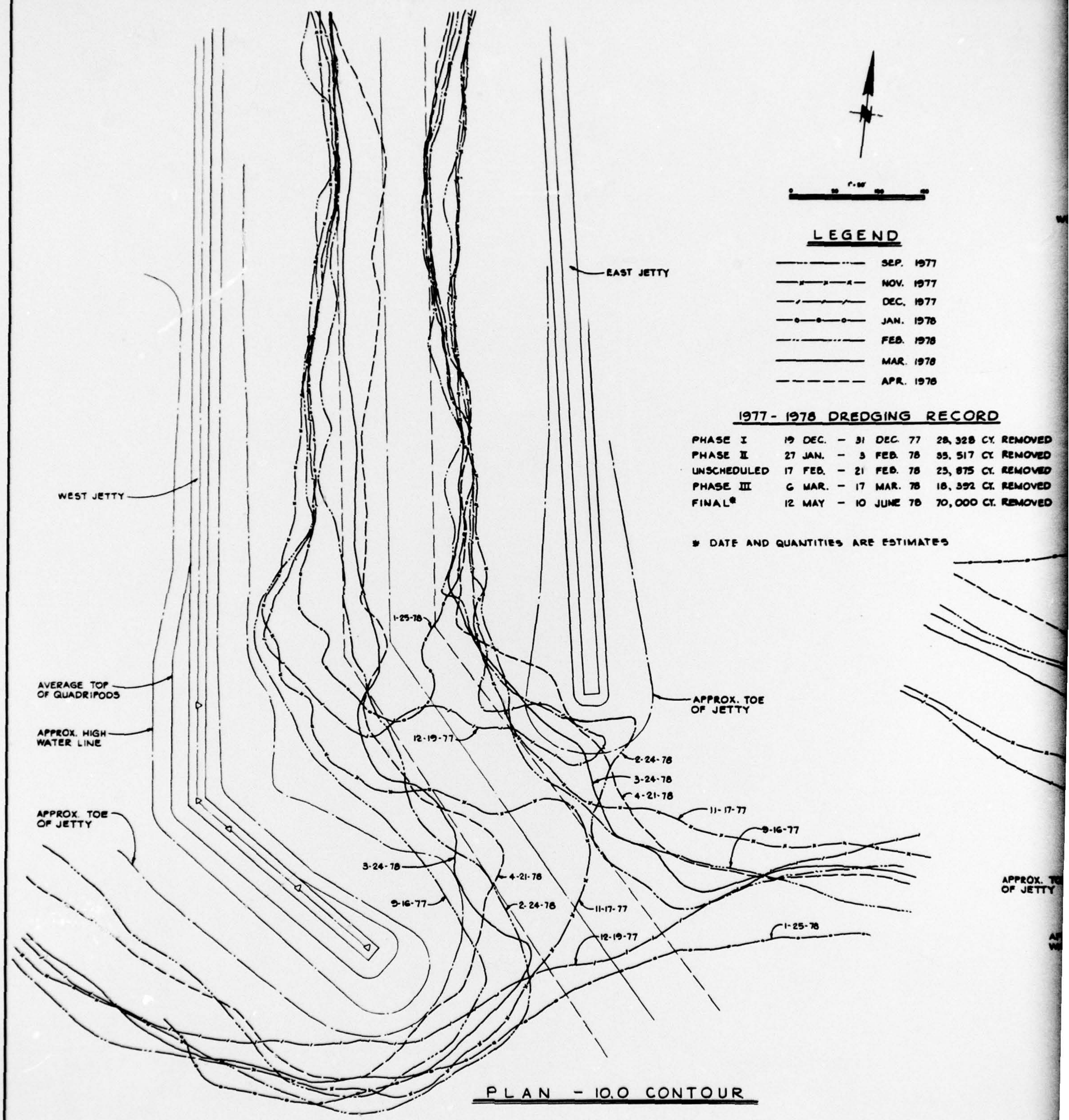
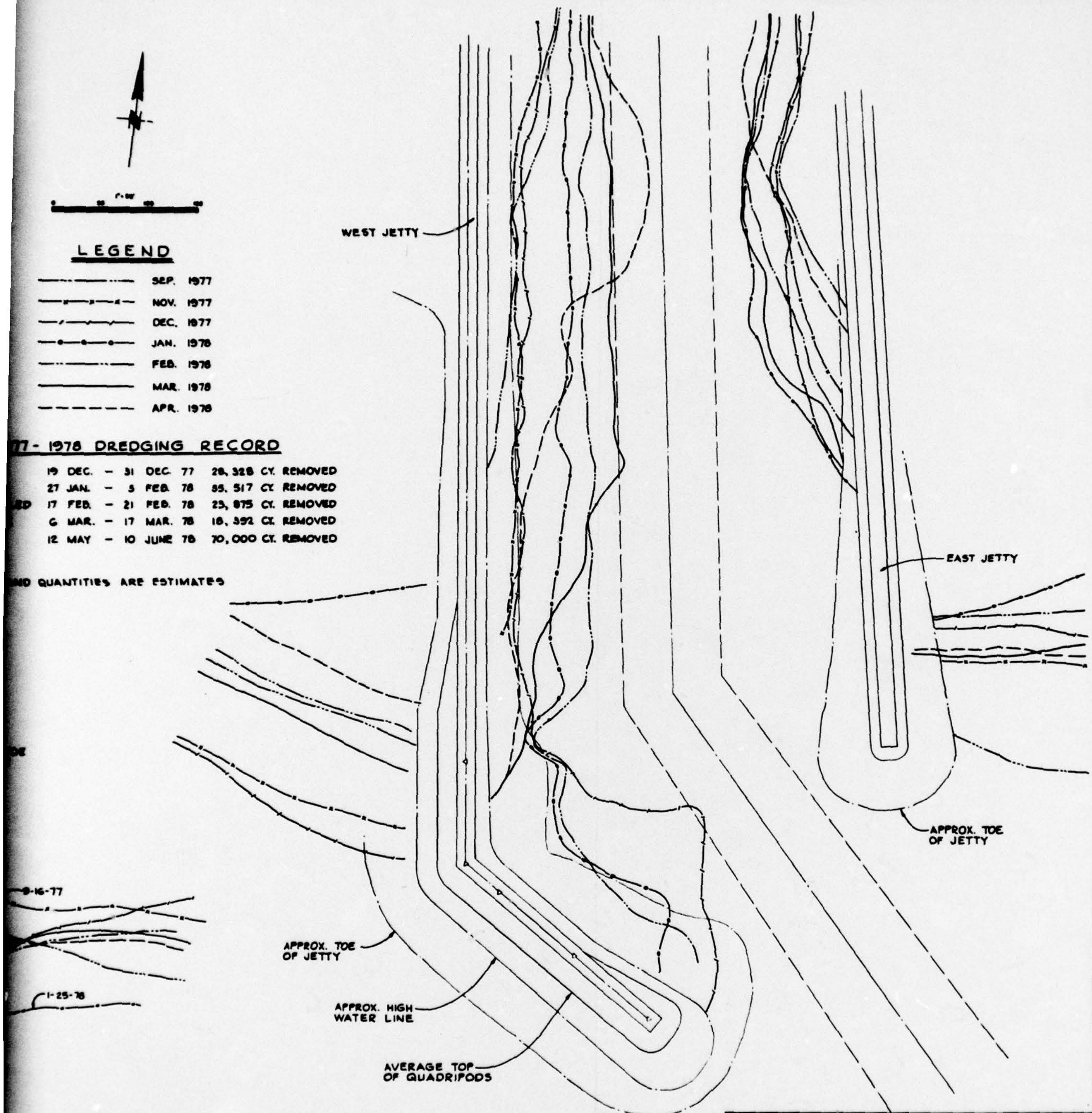


1977 - 1978 DREDGING RECORD

PHASE I	19 DEC.	–	31 DEC. 77	28, 328 CY. REMOVED
PHASE II	27 JAN.	–	3 FEB. 78	35, 517 CY. REMOVED
UNSCHEDULED	17 FEB.	–	21 FEB. 78	23, 875 CY. REMOVED
PHASE III	6 MAR.	–	17 MAR. 78	18, 392 CY. REMOVED
FINAL*	12 MAY	–	10 JUNE 78	70, 000 CY. REMOVED

* DATE AND QUANTITIES ARE ESTIMATES

PLAN - 10.0 CONTOUR



PLAN - 0.0 CONTOUR

SANTA CRUZ HARBOR, CALIFORNIA
SHOALING STUDY

DEPTH CHANGES 1977 - 1978

U. S. ARMY ENGINEER DISTRICT, SAN FRANCISCO

Prepared & Submitted By Moffatt & Nichol, Engineers Long Beach, California	To Accompany Shoaling Study Report Dated: June 1978

PLATE 10

2